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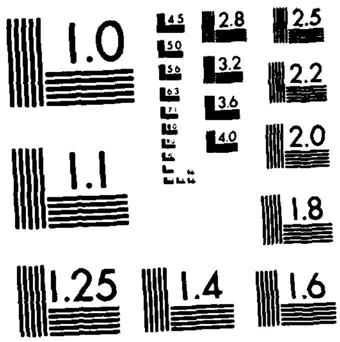

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Monterey, California

THESIS

METHODS OF HYDROGRAPHIC SURVEYING USED
BY DIFFERENT COUNTRIES

by

Athanasios E. Palikaris

March 1983

Thesis Advisor: G. B. Mills

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Methods of Hydrographic Surveying Used by
Different Countries

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Submitted in partial fulfillment of the
requirements for the degree of

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I. INTRODUCTION

The potential accuracy of the data collected during hydrographic surveys has been a subject of increasing interest and research during recent years. The data collected during a hydrographic survey consists of basically two types: survey vessel position and simultaneous depth determination. The accuracies of the final charted soundings depend on both the positional accuracy and the accuracy of depth measurements. A low positional accuracy makes useless a highly accurate depth measurement, and vice versa. This is particularly true for an uneven bottom where small horizontal displacements result in large differences in the measured depth.

Although positioning and sounding are the two basic operations of a hydrographic survey, they are not the only ones. For a hydrographic survey to be started and completed, many other operations are required. Initially, the hydrographic surveyor has to establish horizontal control consisting of fixed reference points (usually on land) from which he will be able to obtain his vessel's position. Secondly, he must establish a fixed reference plane (sounding datum) to which measured depths will be referenced. This is necessary because the sea surface is not fixed but is subject to vertical fluctuations due to wind and tides.

The present study examines only hydrographic surveys conducted for the purpose of compiling nautical charts for the safety of modern navigation. For this purpose the International Hydrographic Organization (IHO) states some minimum recommended accuracies that should be attained during the hydrographic surveys. These are published in IHO Special Publication 44, "Accuracy Standards Recommended for Hydrographic Surveys" which has recently been revised (December 1982). These recommendations of the IHO provide the framework for this study.

Specifications and procedures as well as methods for hydrographic surveying which have been adopted by different countries are examined and compared with each other as well as with those recommended in the IHO standards. More specifically, the objectives of this thesis are twofold. The primary aim is to provide suggestions and references to the Hellenic Navy Hydrographic Service for development of a Greek Hydrographic Manual, especially in the areas of horizontal control and hydrographic surveying. Secondly, it will help make other hydrographers aware of some of the unique methods used by other hydrographic organizations.

II. HORIZONTAL CONTROL

A. GENERAL CONSIDERATIONS

Horizontal control for hydrography is often based on preexisting geodetic control. When it is unavailable or insufficient the hydrographer must establish his own horizontal control network or supplemental control stations. The accuracy requirements for horizontal control for hydrography are not as strict as those for land surveys. The IHO Special Publication No. 44 suggests some minimum accuracy standards and gives some general specifications in order to achieve these standards. Most of the member countries of IHO have devised their own standards and specifications which are more detailed than those recommended in S.P. No. 44. For horizontal control the IHO recommended standards of accuracy are [Ref. 1]:

"(1) Primary shore control points should be located by survey methods at an accuracy of 1 part in 10,000. Where the survey is extensive, a higher degree of accuracy must be adopted to ensure that the relative positions are in error by not more than half the plottable error at the scale of the survey.

(2) When satellite positioning is used to determine the location of shore stations, ties should be made to the local horizontal datum.

(3) Where no geodetic control exists, a point of origin for the horizontal control should be determined by astronomical observations or satellite positioning, the probable error of which should not exceed 2" of arc or about 60 meters.

(4) Secondary stations, required for local positioning (usually visual) which will not be used for extending the control, should be located such that the error does not exceed the plottable error at the scale of the survey (normally 0.5 mm on paper)."

Before proceeding to the specific procedures and methods used by different hydrographic services, some preliminary comments should be made. The meaning of the term probable error is a well defined term in probability and statistics. It is a plus or minus quantity that may be larger or smaller than the resultant error, and its probability of being larger is equal to its probability of being smaller that is 50% probability [Ref. 2]. There seems to be a difference of opinion between various users of the IHO S.P. 44 as to whether the intended meaning of the term in the publication is the above mentioned one or not. Another controversial term is the term "plottable error" which is not defined anywhere in the literature. It may be interpreted to mean the smallest positional error[1] that the human eye can detect through visual inspection of a graphic product - a chart, map or hydrographic field sheet - about 0.5 mm. Horizontal control surveys may be divided according to the methods of execution: (1) ground survey which include triangulation, traverse, and trilateration, (2) satellite

[1]Positional Error: The amount by which a cartographic feature fails to agree with its true position [Ref. 3].

methods, and (3) photogrammetric methods. The required accuracy of the horizontal control between stations is independent of the method of survey.

B. THE U.S. NATIONAL OCEAN SURVEY (NOS) METHODS AND PROCEDURES FOR ESTABLISHING HORIZONTAL CONTROL

The U.S. NOS's methods and procedures for establishing horizontal control are of particular interest. They are very straightforward and unambiguous, which is very important for the inexperienced surveyor. In the United States the governmental agency responsible for the establishment and maintenance of the basic horizontal (and vertical) geodetic control is the National Geodetic Survey (NGS), a component of the NOS, which is the same agency responsible for the hydrography of the U.S. waters.

Horizontal control in the United States is classified as first, second and third order according to the relative accuracy between directly connected adjacent points.

Classification	Accuracy
1st Order	1 part in 100,000
2nd Order Class I	1 part in 50,000
2nd Order Class II	1 part in 20,000
3rd Order Class I	1 part in 10,000
3rd Order Class II	1 part in 5,000

Horizontal control for hydrographic surveys must meet 3rd Order Class I or 2nd Order Class II accuracies. Lower accuracies are permitted for some secondary stations which will not be used to extend the control (such as visual signals).

The two main methods of establishing horizontal control are triangulation and traverse. Triangulation is a method of surveying in which the stations are points at the vertices of a network of triangles. The angles of the triangles are measured instrumentally and the sides are derived by computation from selected triangle sides called bases (for base lines), the lengths of which are obtained from direct measurements [Ref. 4]. Traverse is a method of surveying in which a sequence of lengths and directions of lines between points on the earth are obtained from field measurements and used in determining positions of the points [Ref. 5]. Trilatelation[2], is a third possible method but is rarely used in establishing control for hydrography. NOS has developed many detailed specifications to meet the required standards of the IHO. Nevertheless, as it is stated in the NOS specifications [Ref. 7]:

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[2]Trilateration: A method of surveying in which the lengths of the triangle sides are measured, usually by electronic methods and the angles are computed from the measured lengths [Ref. 6].

"Although an absolute guarantee cannot be given that a particular standard will be met if all stated specifications are followed, it is reasonably certain that the closures in length and position will be about one-half of those stated for a particular standard."

Table XVII of Appendix A shows the "Classification, Standards of Accuracy and General Specifications for Horizontal Control". Of particular interest are the detailed observational procedures and checks for the various orders. The most important specifications for 3rd Order Class I accuracy (which is the one most commonly used by the hydrographer) are mentioned here. Appendix A provides the whole set of the NOS specifications together with some additional clarifications and examples.

For the observation of horizontal angles, either for traverse or for triangulation, four plate settings are required. Each measured angle for each plate setting has to be observed with two positions of the telescope commonly called direct and reverse or circle left and circle right. Angles at any plate setting should not differ more than 5" from the mean reading for all settings. The measuring instrument should be capable of being read directly to 1" of arc.

1. Triangulation

For triangulation, the average triangle closure (Appendix A) should not exceed 3" while the maximum closure should seldom exceed 5". The strength of figure R is a mathematical tool employed by the U.S. NOS [Ref. 8] to measure and compare various computational routes in a triangulation network. The best computational route is the one resulting in the least value for R.

The strength of figure R is defined as:

$$R = \frac{D-C}{D} \sum \left(\delta_A^2 + \delta_A \delta_B + \delta_B^2 \right)$$

where D = The number of directions observed, not including the fixed side (starting azimuth).

C = The number of geometric conditions.

$_A$ = The tabular difference for one second in the log sine of angle A in the sixth decimal place.

δ_B = Same as δ_A but for angle B.

= (n' - s' + 1) + (n -2s + 3)

where n = Total number of lines.

n' = Number of lines observed in both directions (including the fixed line).

s = Total number of stations.

2. <u>Traverse</u>

Traverse is the main method used by the hydrographic surveyor to establish horizontal control. For 3rd Order Class I traverses, the NOS specifications give the following closure limits:

angular closure: 3" per station or $10''\sqrt{N}$

distance closure: $0.4\text{m}\sqrt{K}$

where: N is the number of angle points.

K is the total distance in kms.

The following additional specifications for 3rd Order Class I traverses are given by the NOS Hydrographic Manual:

(1) "Station spacing must be between 2 and 5 kms, closer spacing being permitted where the terrain obscures the line of sight. The minimum length of line should seldom be less than 200 m for electro-optical instruments used and 500 m for lines measured by microwave instruments" [Ref. 9].

(2) A position check is required for "wing" or "spur" points not included in the regular traverse. Depending upon geometric configuration and intervisibilities between stations, many different methods can be used. An example of one of those methods, as illustrated in the NOS Hydrographic Manual is shown in Figure 1, where spur point B' is located by observing the angle ABB' and measuring the distance BB'. Angle BAB' is observed and then distance BB' is computed, via the law of sines, and compared to the measured distance.

If three-point sextant fixes are employed for hydrographic positioning control, less accurate traverse methods can be used for the location of stations for visual signals. According to the NOS Hydrographic Manual, whenever traverse methods with less than 3rd Order standards are used, the following requirements should be met [Ref. 10]:

"(1) Total length of traverse must not exceed 2 km.

(2) Traverses with more than two lines shall be closed to within 1 part in 2,500.

(3) These traverses should start from stations of at least 3rd Order Class II accuracy.

(4) Initial azimuths require at least an accuracy of 1 minute of arc.

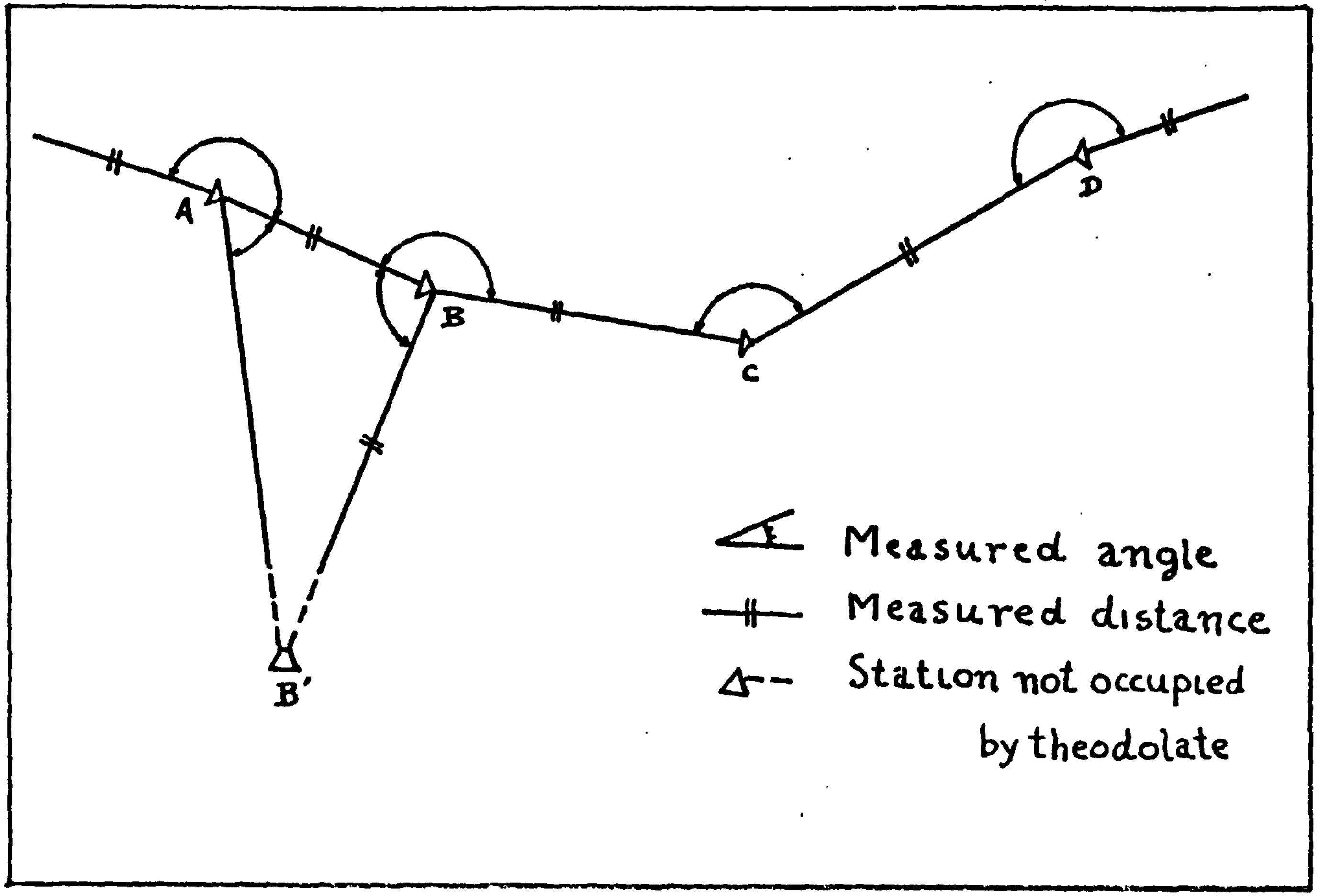

Figure 1. Traverse with a Check "Spur Point"
[From NOS Hydrographic Manual]

(5) Traverse angles and their explements can be measured by one pointing of the instrument, and must close the horizon to within 1 minute of arc.

(6) Distances can be measured by a non-standardized steel tape. Stadia distances[3] should be used as a last resort and only when terrain restrictions prevent the use of steel tape. In these cases, distances should be kept less than 500 m and readings on each of the three wires must be observed and recorded.

(7) Slope corrections to taped distances need not be applied for slopes less than 2°."

3. Other Less Accurate Methods

a. Photogrammetric Methods

These methods utilize aerial photography and are used when ground survey methods are impractical or uneconomical. Two basic methods are used [Ref. 12].

(1) Location by transfer where field identified photo-hydro control stations are directly transferred from a photo to a shoreline manuscript by means of adjacent shoreline pass points shown in the photos and on the manuscript.

(2) Location by radial line intersection where points shown on at least two overlapping photos are transferred onto a shoreline manuscript.

b. Sextant Methods

These methods are occasionally used to supplement existing control. Three basic methods are used:

---

[3]Stadia distance is a rapid indirect method of distance determination. A vertical, graduated rod is observed by a special optical instrument (level or theodolite) and the intercept subtended by a known small angle determines the distance. The small known angle is usually defined by two horizontal wires in the reticle of the telescope above and below the wire of the optical axis [Ref. 11].

(1) Location by strong three point fixes at the station with check angles to a fourth station (sextant resection).

(2) Location by fixing the position of the survey vessel by strong three-point fixes and simultaneous sextant cuts to the unknown station (Figure 2). In this method the vessel stays stationary at point S_1 so that a good three-point fix can be obtained, by measuring the angles between A and B, B and C and the unknown station "a" and any of the other signals (A, B or C). The above process is repeated with the vessel being at positions S_2 and S_3. Station "a" is located from the three cuts from the established positions of the vessel at S_1, S_2 and S_3.

(3) Location by intersection of sextant cuts observed from three or more existing control stations. The angles are measured from other known control stations to the new stations.

c. Plane Table Methods

These graphic triangulation or traverse methods are rarely used to supplement existing control since they have mostly been replaced by photogrammetric methods. "The plane table is a field device for plotting the lines of a survey directly from the observations. It consists essentially of a drawing board mounted on a tripod with a ruler on which a telescope or other sighting device is mounted" [Ref. 13]. The NOS Hydrographic Manual gives the following specifications for plane table surveying [Ref. 14]:

"... 90% of the control stations located will be within 0.5 mm of their correct geographic position of the scale of the plane table sheet. No stations shall be in error by more than 0.8 mm. Closing errors of plane table traverses prior to adjustment shall not exceed 0.25 mm/km at the scale of the sheet; and in no case shall the total closing error exceed 2.0 mm."

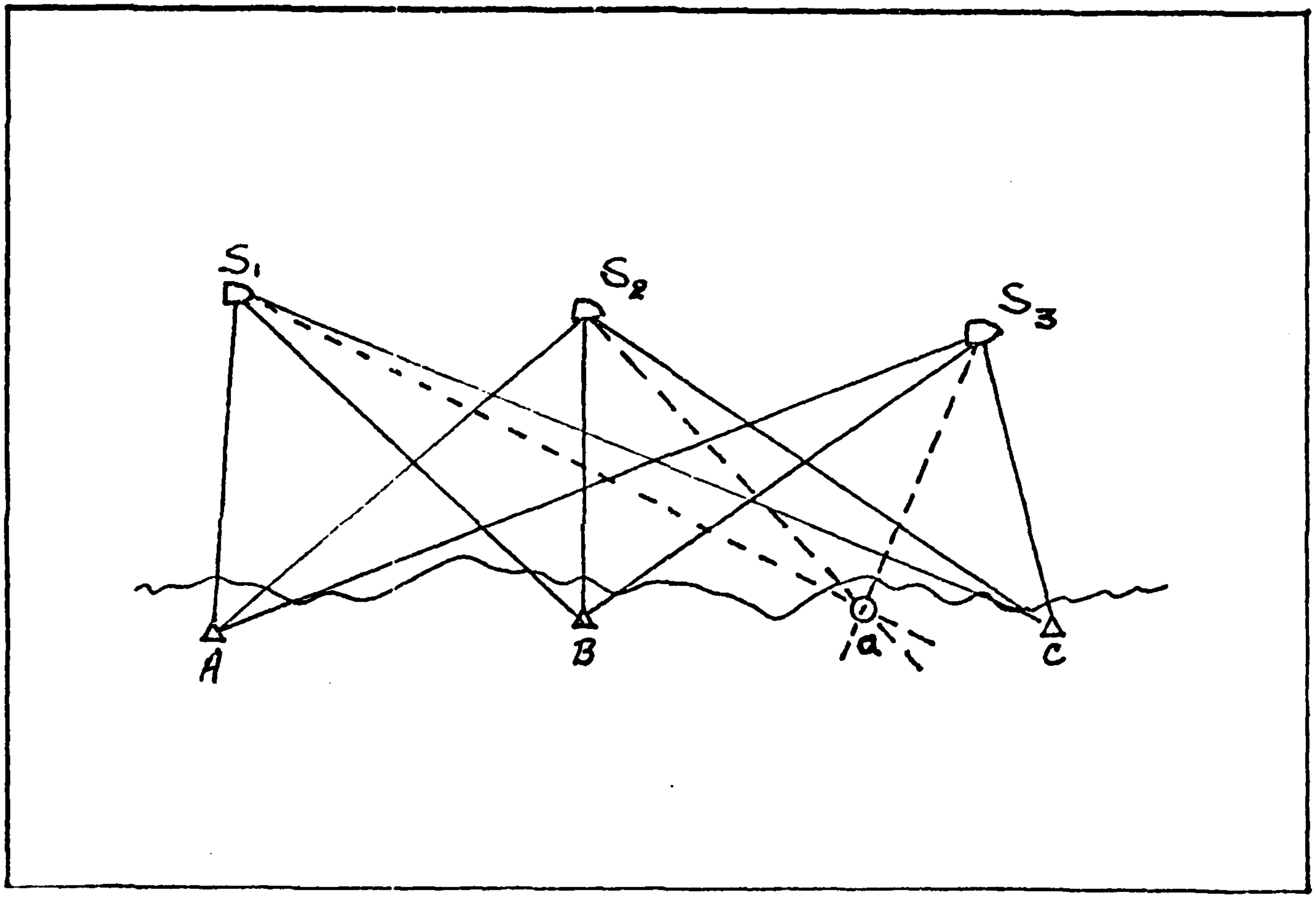

Figure 2. Location of a Horizontal Control Point Ashore from Observations Off-Shore

C. CANADIAN HYDROGRAPHIC SERVICE METHODS AND PROCEDURES FOR ESTABLISHING HORIZONTAL CONTROL

In Canada, horizontal control is classified as first, second, third and fourth order. These classifications are based on the concepts of standard deviation and confidence region and can be used to simplify the design and analysis of a horizontal control network. In all cases the accuracies required by the Canadian Hydrographic Service for primary stations and antennae sites for electronic positioning systems must meet third order accuracy standards [Ref. 15].

1. The Concept of Standard Deviation and Confidence Region

Standard deviation or standard error is a statistical measure of precision. It measures the dispersion of a set of observations of a quantity (such as an angle or distance) from the mean of these observations. The standard deviation, s, of a group of n observations x_1, x_2, x_3, ... x_n is given by the formula:

$$S = \sqrt{\frac{\sum_{i=1}(x_i - \bar{x})}{n-1}}$$

where $\bar{x}$ is the mean of all the observations

$$\bar{X} = \frac{\sum_{i=1}^{n} x_i}{n}$$

The number n-1 gives the degrees of freedom of the observations (the first of the n observations establishes an initial value for the measured quantity while the other n-1 observations are redundant).

In surveying, random[4] observational errors are assumed to be distributed according to the normal distribution with standard deviation σ. In this case we expect 68.27% of the observations to lie within one standard deviation of the mean (1σ), and 95.45% within two standard deviations (2σ). For two dimensional errors (such as positioning) the two-dimensional normal distribution function is used and the resulting error is an ellipse [Ref. 16]. The standard error ellipse is the one based on the standard deviation of unit weight -- the two lines of position are equally weighted [Ref. 17].

A confidence region is defined as a region within which we have a specified degree of confidence (expressed as a percentage) that an actual value lies. For normally distributed observations in two dimensions a confidence region is bounded by an ellipse. Figure 3 shows a 95% confidence region. The 95% confidence region is an enlargement of the standard error ellipse. A standard error ellipse bounds a confidence region of 30 to 39% depending on the number of redundant measurements (degrees of freedom)

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[4] Random Errors: Those errors whose occurance depends on the law of chance only [Ref. 18].

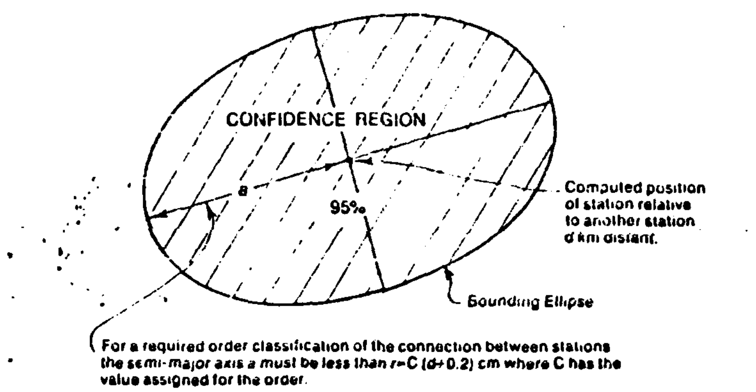

Figure 3. Ellipse Showing the 95% Confidence Region of One Station Relative to Another
[From the Canadian Specifications for Control Surveys]

[Ref. 19]. The axes of the 95% confidence region are obtained by multiplying the corresponding axes of the standard error ellipse by an appropriate factor given in Table I. This factor depends on the number of degrees of freedom used to determine the standard error. Assuming that good estimates of standard errors of measurements are available, the factor 2.45 corresponding to infinite degrees of freedom should be used; otherwise the appropriate factor from Table I (for the corresponding number of degrees of freedom) has to be used. Inspecting Table I one observes that the larger the number of the degrees of freedom or observations, the closer the factor C_{95} comes to the value of 2.45. All factors of Table I have been derived from the F distribution which refers to the distribution of the ratio of the variances of two independent random samples [Ref. 20]. Appendix B includes tables with typical values for standard errors for various instruments and methods of observation.

2. Classification of Horizontal Control Surveys

The order of accuracy of a horizontal control station in Canada is determined by comparing the semimajor axis of the 95% confidence region of the position of the station with respect to any other station, to the value:

$r = c(d + 0.2)$

TABLE I

FACTORS FOR CONFIDENCE REGIONS

f	C_{90}	C_{95}	C_{99}
1	9.95	19.97	99.99
2	4.24	6.16	14.07
3	3.31	4.37	7.85
4	2.94	3.73	6.00
5	2.75	3.40	5.15
6	2.63	3.21	4.67
7	2.55	3.08	4.37
8	2.50	2.99	4.16
9	2.45	2.92	4.00
10	2.42	2.86	3.89
11	2.39	2.82	3.80
12	2.37	2.79	3.72
13	2.35	2.76	3.66
14	2.34	2.73	3.61
15	2.32	2.71	3.57
16	2.31	2.70	3.53
17	2.30	2.68	3.50
18	2.29	2.67	3.47
19	2.28	2.65	3.44
20	2.28	2.64	3.42
25	2.25	2.60	3.34
30	2.23	2.58	3.28
40	2.21	2.54	3.22
50	2.20	2.53	3.19
60	2.19	2.51	3.16
80	2.18	2.49	3.12
120	2.17	2.48	3.09
∞	2.15	2.45	3.03

NOTES:
f = degrees of freedom in the adjustment
C_{95} = factor by which axes of standard ellipse are to be multiplied to obtain 95 percent confidence region

[From the Canadian Specifications for Control Surveys]

where: r is expressed in centimeters

d is the distance to any station in kilometers

c is a factor assigned for the order of accuracy.

The values of c for the various orders of accuracy are listed in Table II. For two stations which are 10 km apart to be classified as first order (c = 2), the semimajor axis of the 95% confidence region of one station relative to the other must be less than or equal to 20.4 cm [2 x (10 + 0.2)]. Figure 4 is a graph of r against distance d, for the values of c assigned to various orders of survey.

TABLE II

VALUES OF C FOR HORIZONTAL CONTROL SURVEYS ACCORDING TO ORDER

Order	c
1st	2
2nd	5
3rd	12
4th	30

The peculiarity of the Canadian classification is that the relative accuracy between any two stations of a network of a specific order, expressed as a ratio of their distance, is different for different distances (Table III). This peculiarity occurs because the Canadian classifications account for the fact that the errors in relative accuracy

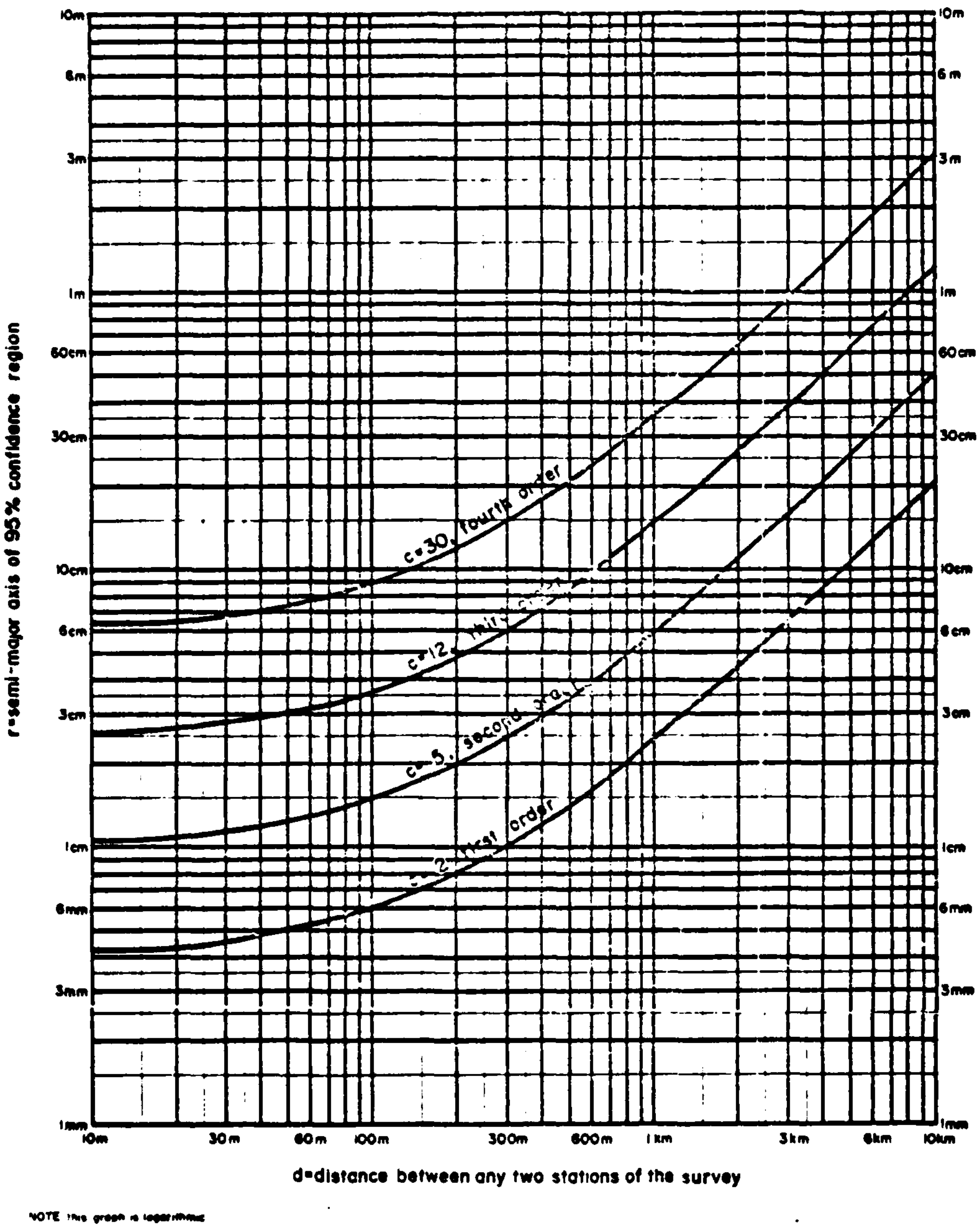

Figure 4. Accuracy Standards for Horizontal Control Surveys (based on r = C(d + 0.2), where r is in cm and d in km) [From the Canadian Specifications for Control Surveys]

TABLE III

ACCURACY STANDARDS FOR HORIZONTAL CONTROL SURVEYS

(showing the variation in proportional accuracy over short distances)

ORDER	C	SEMI-MAJOR AXIS OF 95% CONFIDENCE REGION, r=C(d+0.2); WHERE d IS THE DISTANCE BETWEEN ANY TWO STATIONS																	
		for d=0.03 km			for d=0.1 km			for d=0.3 km			for d=1.0 km			for d=3.0 km			for d=10 km		
		cm	ppm	ratio	cm	ppm	ratio	cm	ppm	ratio	cm	ppm	ratio	cm	ppm	ratio	cm	ppm	ratio
1	2	0.5	153	1/6500	0.6	60	1/16700	1.0	33	1/30000	2.4	24	1/41700	6.4	21	1/46900	20	20	1/50000
2	5	1.2	383	1/2600	1.5	150	1/6700	2.5	83	1/12000	6.0	60	1/16700	16.0	63	1/18800	50	50	1/20000
3	12	2.8	920	1/1100	3.6	360	1/2800	6.0	200	1/5000	14.4	144	1/6900	38.4	128	1/7800	120	120	1/8300
4	30	6.9	2300	1/430	9.0	900	1/1100	15.0	500	1/2000	36.0	360	1/2800	96.0	320	1/3100	300	300	1/3300

[From the Canadian Specifications for Control Surveys]

are of two types, those proportional to distance and those independent of distance. For lines shorter than 3 kilometers the dominant errors are those that are independent of distance while for longer lines the errors proportional to distance become dominant. The Canadian method of classification of horizontal control has the main advantage that the concept of confidence region permits the prediction of the accuracy of a prospective survey. The design of the survey can be changed to increase the probability of success.

The following simple example shows how the accuracy of a point can be roughly estimated in the design and planning stage of a survey, if some a priori estimation of errors are available. Figure 5 refers to the location of a point B with respect to point A using azimuth and distance measurements. For a rough estimation of the accuracy of point B an approximation of the measured distance AB is required. Let the distance be 1000 meters measured with a technique having a standard error of (1 cm $\pm$ 3 ppm) and the azimuth measured with a technique having a standard error of 5" of arc. The two axes of the 95% error ellipse are determined separately by the methods described in the previous section. The greater of these two axes is the semimajor axis that will determine the order of accuracy.

In this example, the semi-axis in the direction AB is $2.45 \times \sqrt{0.01^2 + (3 \times 1000 \times 10^{-6})^2} = 0.026$ m

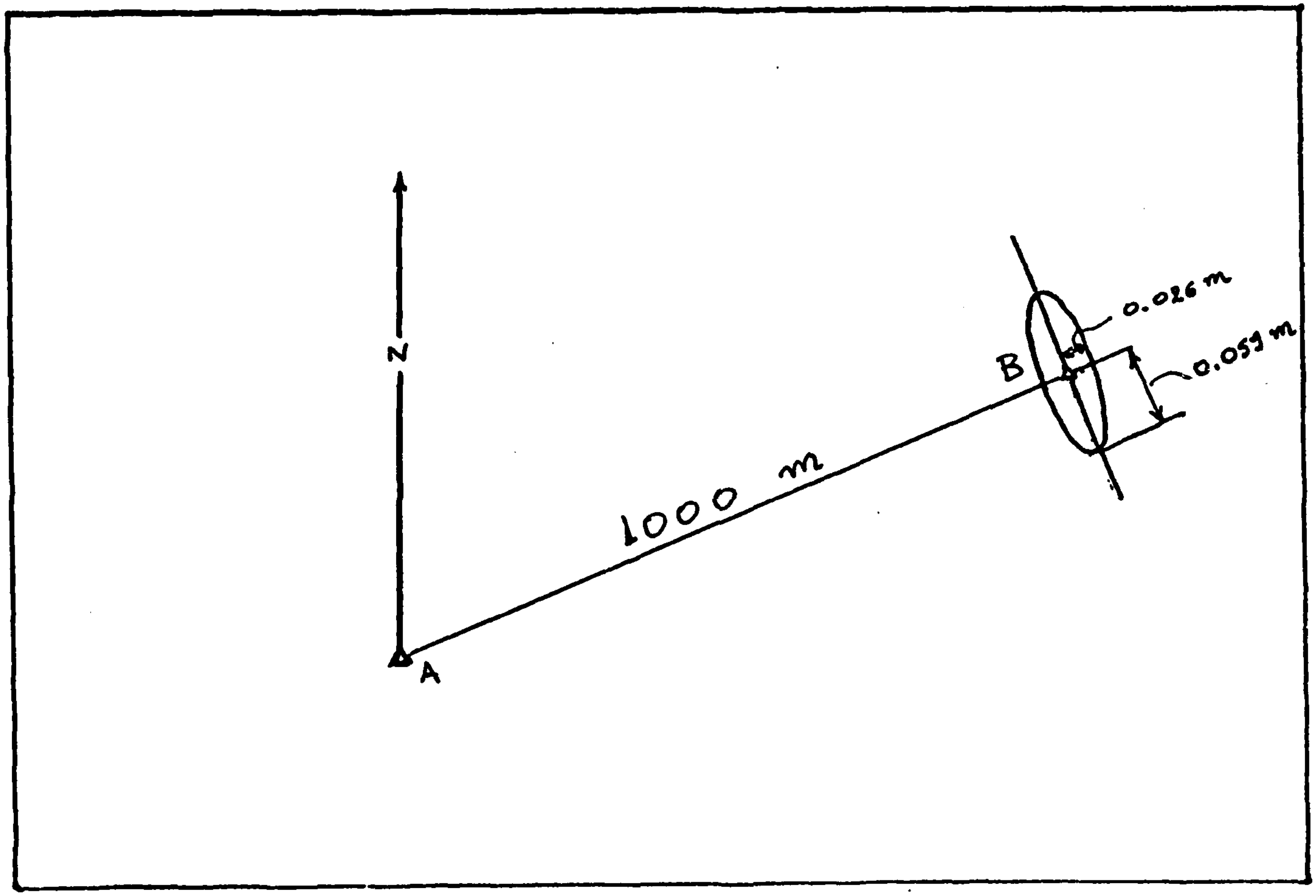

Figure 5. Simple Survey Tie with Azimuth and Distance

while the semi-axis in the direction perpendicular to AB is 2.45 sin 5" x 1000 = 0.059 m. So the semi-major axis is 0.059 m and from the graphs of Figure 4 we see that the accuracy of point B relative to point A is a little better than second order. Similar simple procedures can be used for more complex configurations, examples of which are presented in the Canadian Specifications for Control Surveys [Ref. 21].

3. <u>Measurement and Check Guidelines</u>

For the classical methods of triangulation, traverse and trilateration, the Canadian specifications suggest some measurement and check guidelines summarized in Table IV. For a horizontal control network to be strong and reliable, the stations should be as evenly spaced as possible and adjacent points in the network should be connected by direct measurement, whenever possible. The ratio of the longest length to the shortest should not be greater than five and preferably should be much less.

The guidelines of Table IV are based on experience and the results of analysis of idealized networks like those in Figures 6 and 7. In hydrographic surveys the CHS uses second, third and fourth order standards. The average length per leg for, second, third and fourth order networks are 15 km, 10 km and 5 km respectively.

TABLE IV

MEASUREMENT AND INTERNAL CHECK GUIDELINES

(for long lines only)

		CHARACTERISTICS	ORDER 1st	2nd	3rd	4th
MEASUREMENTS	DIRECTIONS	Least count of theodolite	0.2"	1.0"	1.0"	10.0"
		Minimum number of sets	24(4x6)	6	2	2
		Standard deviation (mean of all sets)	0.7"	2.0"	4.0"	8.0"
	LENGTHS	Standard deviation (complete determination)				
		•Triangulation and Traverse	4 ppm	10 ppm	20 ppm	40 ppm
		•Trilateration	3 ppm	5 ppm	10 ppm	20 ppm
	AZIMUTHS	Minimum number of sets	32	12	4	4
		Standard deviation (mean of all sets)	0.4"	1.5"	3.0"	6.0"
		Maximum number of courses between control azimuths				
		•Triangulation	10	12	15	20
		•Trilateration	6	8	10	12
		•Traverse	5	6	9	12
CHECKS	DIRECTIONS	Standard deviation of a triangle misclosure	1.2"	5.0"	10.0"	20.0"
	AZIMUTHS	Maximum difference between control azimuths	$2''\sqrt{N}$	$5''\sqrt{N}$	$10''\sqrt{N}$	$20''\sqrt{N}$

[From the Canadian Specifications for Control Surveys]

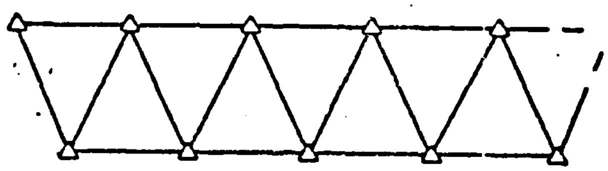

Figure 6. Triangulation: Single Chain Network

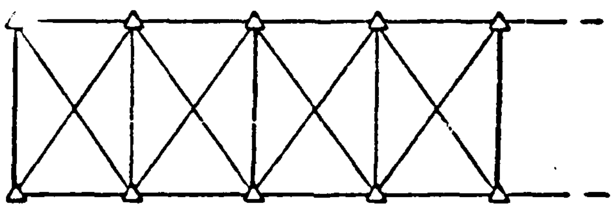

Figure 7. Triangulation: Cross Braced Quadrilateral Network

For triangulation, the suggestions of Table IV are based on the study of single chain network (depicted in Figure 6) as well as that of cross braced quadrilateral networks (depicted in Figure 7). The single chain network is that in which only two sides of each triangle are common to other triangles in the chains (one with the preceding triangle and one with the following one (Figure 6)). As for most triangulation methods all angles in the network have to be observed. For third and fourth order triangulation, one side of every fourth triangle in a single chain network must be measured while for a braced quadrilateral network, one side of every second quadrilateral has to be measured.

For a traverse, the idealized configuration is that of a straight line. For third order accuracy, an azimuth check is required every nine legs. For third order accuracy the azimuth check (maximum permissible angular closure) is $10'' \sqrt{N}$ where N is the number of angles.

D. BRITISH HYDROGRAPHIC DEPARTMENT METHODS AND PROCEDURES FOR ESTABLISHING HORIZONTAL CONTROL

In Great Britain, as in some other countries, the governmental agency responsible for the national geodetic control net is independent from the agency responsible for the hydrography. Again, traditional methods of triangulation and traverse are mainly used for the establishment of the horizontal control.

As far as observational procedures and design of the horizontal control survey, it seems that except for some precise specifications, there is much flexibility for the hydrographic surveyor. "Common sense and judgement must be used in deciding exactly what to do in a particular case" [Ref. 22].

The required accuracy of horizontal control surveys for hydrography is not clearly specified in any of the sources researched. However, it is stated in the General Instructions for Hydrographic Surveyors [Ref. 23] that " ...Hydrographic surveyors..., seldom, even at best, work in the field to an accuracy greater than the Ordnance Survey third order (1 part in 20,000). More normally it equates to fourth order[5]". Pure trilateration methods are very rarely, if at all, used for hydrographic surveys. "To the hydrographic surveyor, trilateration is likely to be of most use in strengthening weak points in triangulation and providing additional checks on the angular measurements." [Ref. 24]

1. Measurement Techniques

For the observation of horizontal angles with a theodolite, two methods are used:

(1) The most commonly used is the direction method [Ref. 25] which involves observations with four plate settings (four zeros) with two positions of the

[5]Fourth order accuracy is defined as that which is less than third order.

telescope for each plate setting. Indeed, this is the same as the previously mentioned NOS method for 3rd Order Class I.

(2) The other method used by the British Hydrographic Office for observing horizontal angles by theodolite, is the repetition method [Ref. 26]. This method requires a special repeating theodolite[6] and is more time consuming than the direction method. In the repetition method, the measured angle is observed at least six times (repetitions). After each measurement (except the last one) the horizontal plate is shifted by the amount of the measured angle, so that each reading is a integer multiple of the measured angle. The difference between the last and the first readings divided by the number of repetitions gives the measured angle. The Admiralty Manual gives detailed guidelines for a complete observation by the repetition method.

For distance measurements either steel tape or electronic distance measuring (EDM) instruments are suggested. Other less accurate methods for distance measurements are occasionally used including tachymetry[7] and subtense bar[8]. Potential accuracies for these methods are listed in Appendix B.

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[6]Repeating Theodolite: A theodolite so designed that successive measures of an angle may be accumulated on the graduated circle and a final reading of the circle made which represents the sum of the repetitions [Ref. 27].

[7]Tachymetry: A method of surveying for the rapid determination of distance (also direction and relative elevation) of a point, with respect to the instrument station by a single observation on a rod or other object at the point. The stadia method of surveying is an example of tachymetry [Ref. 28].

[8]Subtense Bar: A horizontally held bar of precisely determined length, used to measure distances by observing the angle it subtends at the distance to be measured [Ref. 29].

2. <u>Triangulation</u>

The Admiralty Manual of Hydrographic Surveying (AMHS) suggests the following rules of thumb for the design of a triangulation horizontal control survey. These rules are based on experience and the fact that the accuracy of the established points depends to a great extent on the geometrical figures by which they are connected to other points in the scheme. It must be possible to work through the triangulation by two separate routes in order to be able to obtain a check.

The best possible figures for triangulation, according to the AMHS, are shown on Figure 8 and are:

(1) The single triangle (Figure 8a). In this case errors in one triangle are propagated to the triangles that follow it. No receiving angle should be less than about 40° unless one of the sides containing it can be measured.

(2) Triangle with a control station (Figure 8b). The central station D does not strengthen the figure; this figure simply involves shorter sides.

(3) The braced quadrilateral (Figure 8c). This case where both diagonals have been observed is the strongest figure. Angles marked with "x" must not be less than 35° unless a side (preferably the diagonal) is measured or the small angle is measured more accurately by the repetition method.

(4) The quadrilateral with central station (Figure 8d). This figure is not as strong as the braced quadrilateral but it is easier to observe. Observed angles marked by "o" must not be less than 40° or greater than 140°.

(5) The polygon with central station (Figure 8e). This figure is weaker than the braced quadrilateral but easier to observe. A regular pentagon is the best figure of this type. Figures with more than six sides

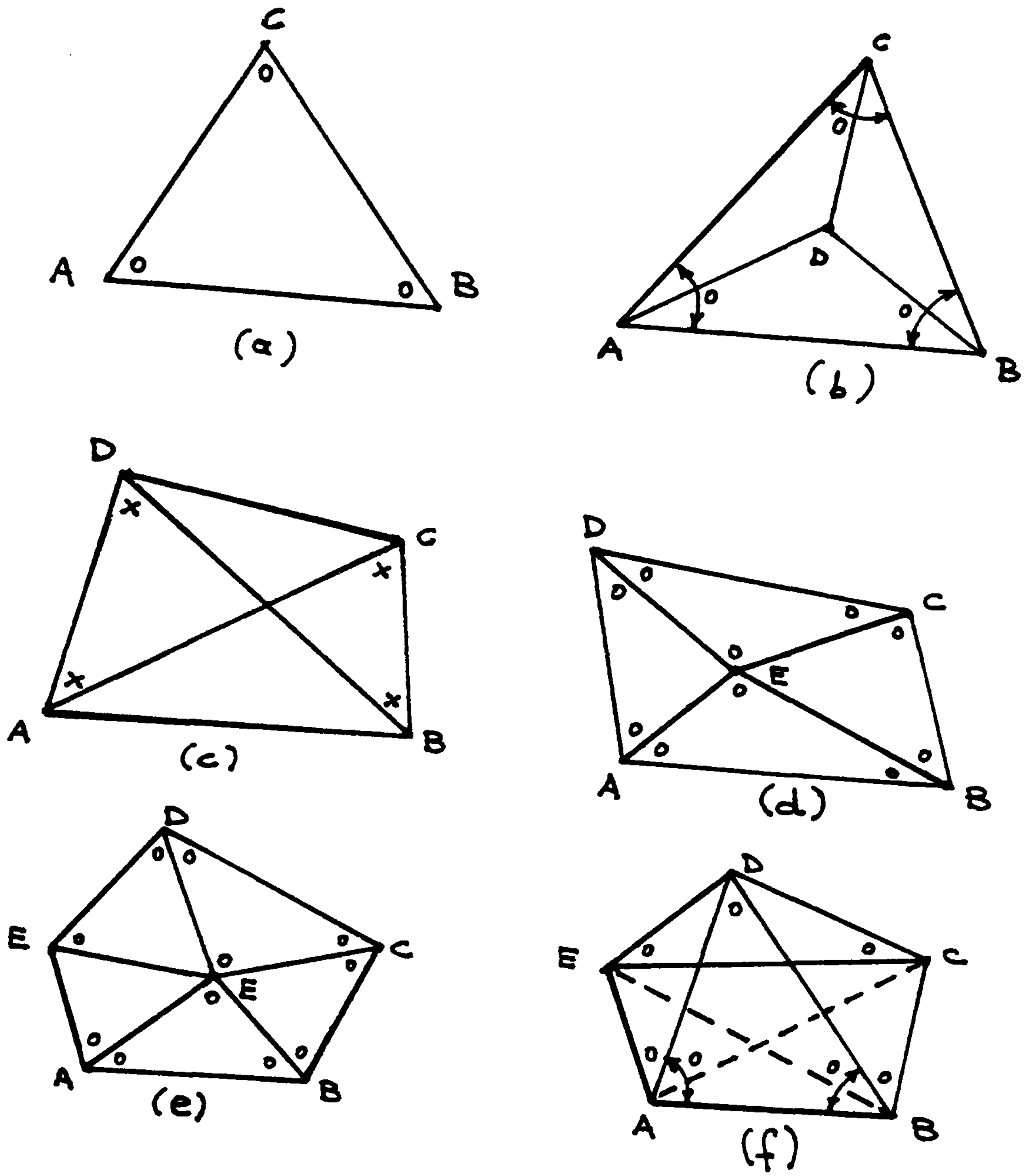

Figure 8. Triangulation Configurations
[From the AMHS]

get weaker and should be avoided. Observed angles marked by o must not be less than 40° or greater than 140°.

(6) The polygon without a central station (Figure 8f). This figure is not strong unless four diagonals are observed when it degenerates into two overlapping quadrilaterals. This configuration of overlapping figures is very strong but should be avoided because its adjustment is too laborious and complicated to be used for hydrographic surveys.

The specifications for triangulation surveys given in the General Instructions for Hydrographic Surveyors (GIHS) require four plate settings for angular measurements but relax the requirements for triangle closures (compared with the NOS specifications) -- average closure 6" and maximum closure 10".

3. <u>Traverse</u>

Traverse methods are adopted by the British Hydrographic Department in four different ways. According to the AMHS, traverses used in hydrography can be of one of the following types:

a. Accurate Traverse

An accurate traverse has standards of accuracy equivalent to those of triangulation. It is employed when it is uneconomic or impossible to carry out triangulation. The lines (legs) should be roughly about the average length of a side of triangulation. The angular measurements for accurate traverses are the same as those for triangulation (four plate settings with both positions of the telescope and rejection limit from the mean 5" to 6" with a 1" theodolite) [Ref. 30]. The closure limits for accurate traverses, as given in GIHS #0809, are:

misclosure in distance = (5N + 5K) cms

angular misclosure = 2(N + 1) seconds of arc

where: N = number of legs in traverse.

K = total distance measured in kms.

b. Minor Traverse

For this type of traverse the accuracy criterion is that there should be no plottable error at the scale of the survey. Minor traverses are run between two known points which are not too far apart and are most useful for coastlining[9]. Direction can be measured by a theodolite, sextant or compass. A minor traverse should always be closed to a known point and the maximum allowable misclosure is $0.5\sqrt{L}$ feet, where L is the total traversed distance in feet [Ref. 32].

c. Beach Traverse

A beach traverse is the simplest type, suitable for establishment of control on a long expanse of beach. The method uses the minimum of equipment and although no

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[9]Coastlining is the accurate delineation of the shoreline and coastal features. The coastline is hydrographic surveying is defined as the "high water line" [Ref. 31].

angular measurements are necessary, they may be used at times. All measurements are plotted graphically and the principle used is illustrated in Figure 9. The various lines (legs) of the traverse are equal in length and as far as possible they are all segments of the same straight line measured with a long wire marked every 100 units. A ranging pole[10] is used on the transit[11] of control and turning points so that very sensitive angular control is maintained. If a change in direction has to be made as that at points b and f, an offset distance is measured with the steel tape as the shortest (perpendicular) distance. For higher accuracy, the hypotenuse of the right triangle containing the offset must be longer.

For traverses in general, the best figure is that shown on Figure 10a where the lengths of the various lines (legs) are equal and the angles are each equal to or nearly equal to 180°. In other words, the traverse is a straight line with equally spaced stations. The more the traverse deviates from the straight line and the greater the variation in length of the legs the weaker the traverse will

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[10]Ranging Pole: A long slender rod, as of timber or metal fitted with a sharp pointed steel shoe. It is usually painted red and white alternately and used to line up points of a survey [Ref. 33].

[11]Transit: In a traverse, any point of junction of two legs [Ref. 34].

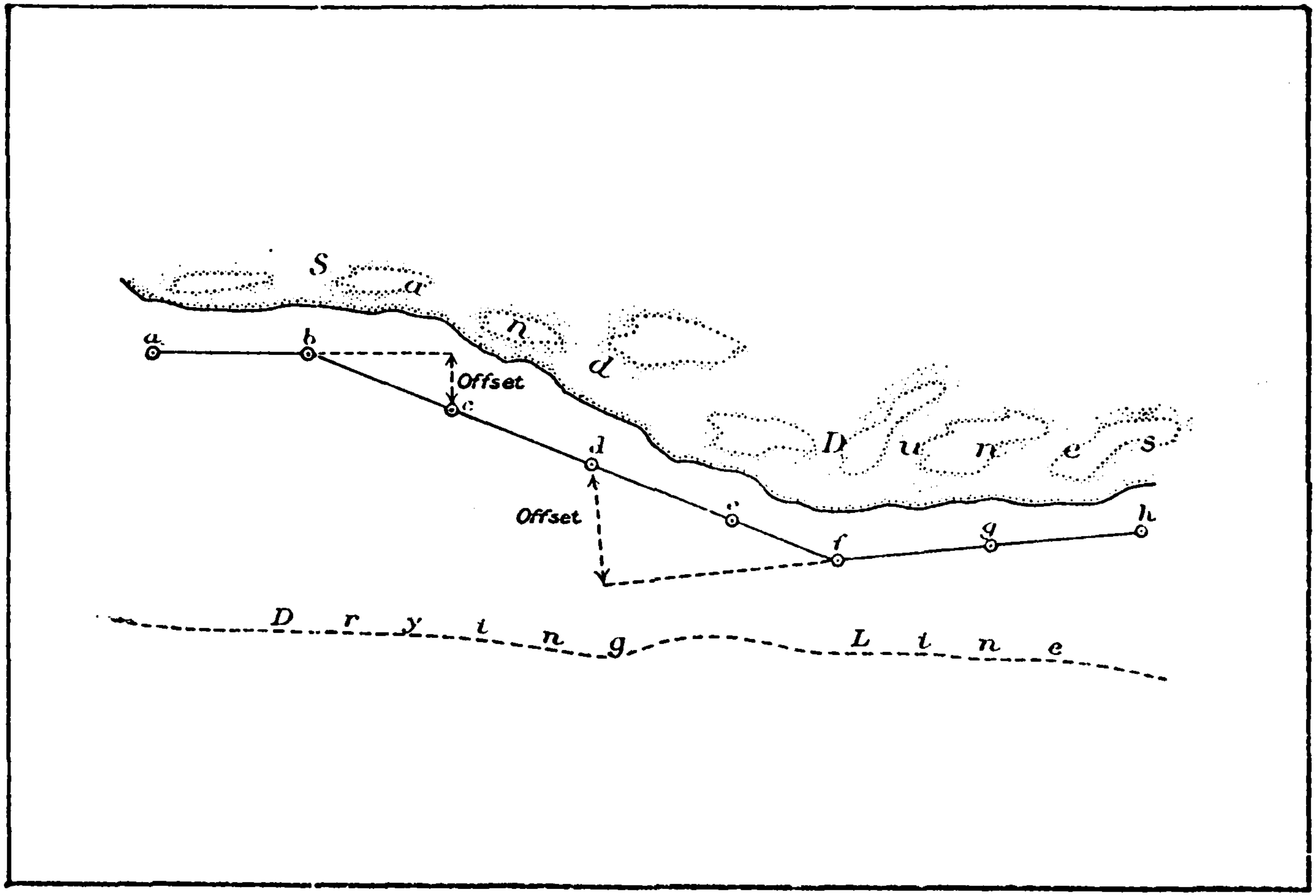

Figure 9. Beach Traverse
[From the AMHS]

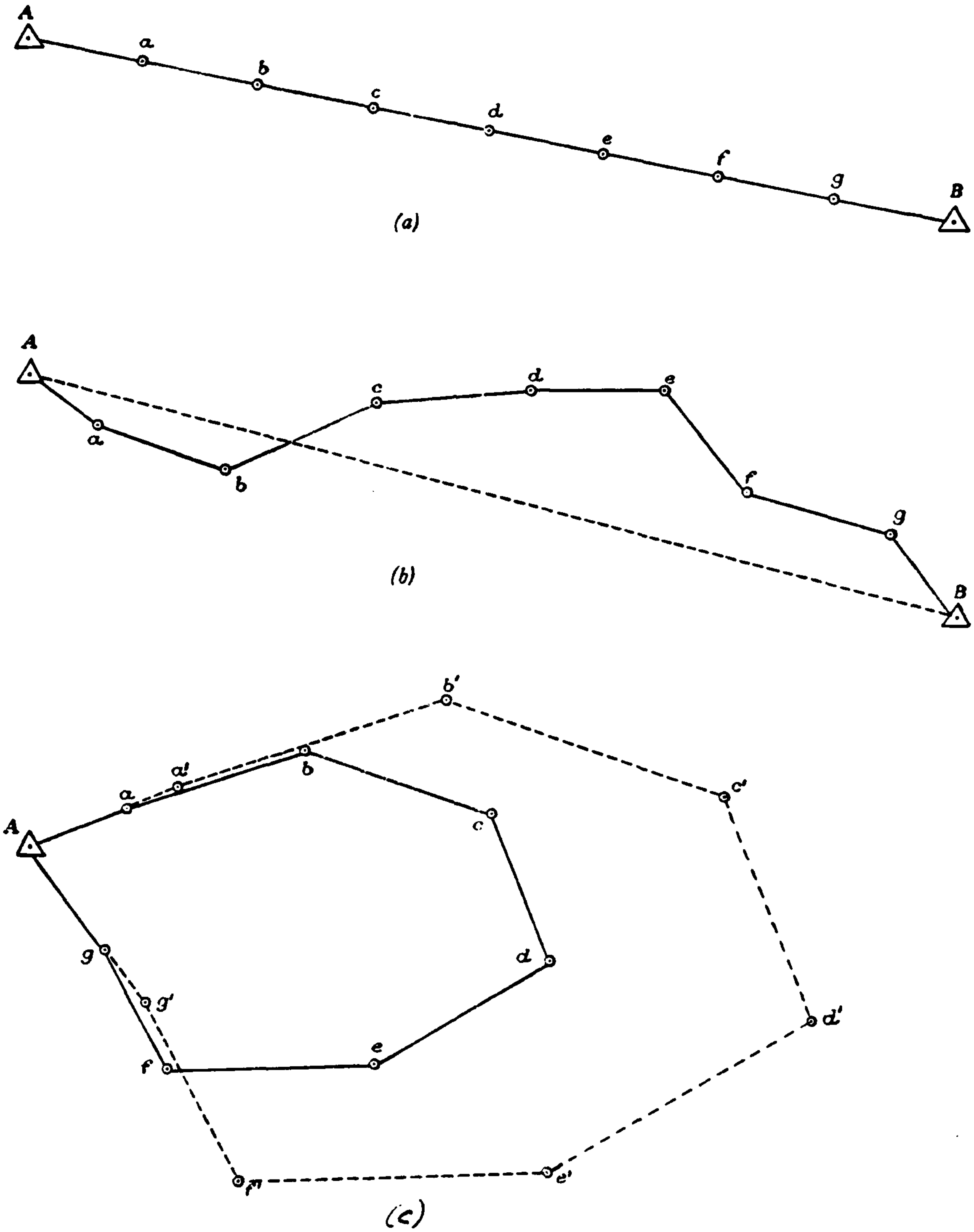

Figure 10. Traverse Configurations
[From the AMHS]

be. In a closed traverse like that of Figure 10c, there is a check on the angular observations but there is none on the linear measurements. Therefore, there is a possibility that an error proportional to distance may occur and not be detected. In that case, there would be perfect closure at the starting point, but the intermediate turning points a, b, c, and so forth would be displaced.

In practice traverses will be of the form shown in Figure 10b, but the more they approach those of Figure 10a the better. Traverses approaching the closed form should be avoided. A useful rule of thumb is that the direct distance between the starting and terminal points should never be less than half the total distance run for the traverse (the sum of the lengths of all legs).

4. Other Less Accurate Methods

Two modifications of the regular methods of triangulation and traverse are suitable for establishing horizontal control for visual signals for hydrographic surveying. Both use marks or stations located at sea.

a. The Use of Temporary Floating Marks With Triangulation

This method is used when it is not possible to measure distances and run traverses due to lack of operable distance measuring equipment or when the terrain hinders the use of a regular triangulation. It can be used to establish secondary stations ashore in different situations, three of which are discussed here. One is shown in Figure 11 where A and I are two already established stations, and G and H are unknown stations. It can be easily seen from the figure that the triangles AIG and GIH are very weak figures because their receiving angles G and H are very large. In this method the ship is anchored first at S_1 so that the triangles AS_1G and IS_1G (or the quadrilateral AS_1IG) form a strong triangulation figure through which station G is established. Since distance AI is known, only simultaneous theodolite angles are observed from points A, G, and I to the foremast. Station S_1 is determined from the triangle AIS_1 in which the base AI is known and the angles at A and I have been measured. Then station G is determined by resection from the known stations A, S_1, I. The fact that the ship, although anchored, is not fixed does not cause any problem provided that the observations are simultaneous. The same procedure is used for station H with the ship anchored in S_2.

Another situation in which this method can be used is in a channel (Figure 12) where on one side there are two known intervisible stations A and B, and on the other side two intervisible stations C and D which have to be established. The channel is too wide for the quadrilateral ABDC to be used. In this situation the ship anchors successively at S_1 and S_2 so that quadrilaterals

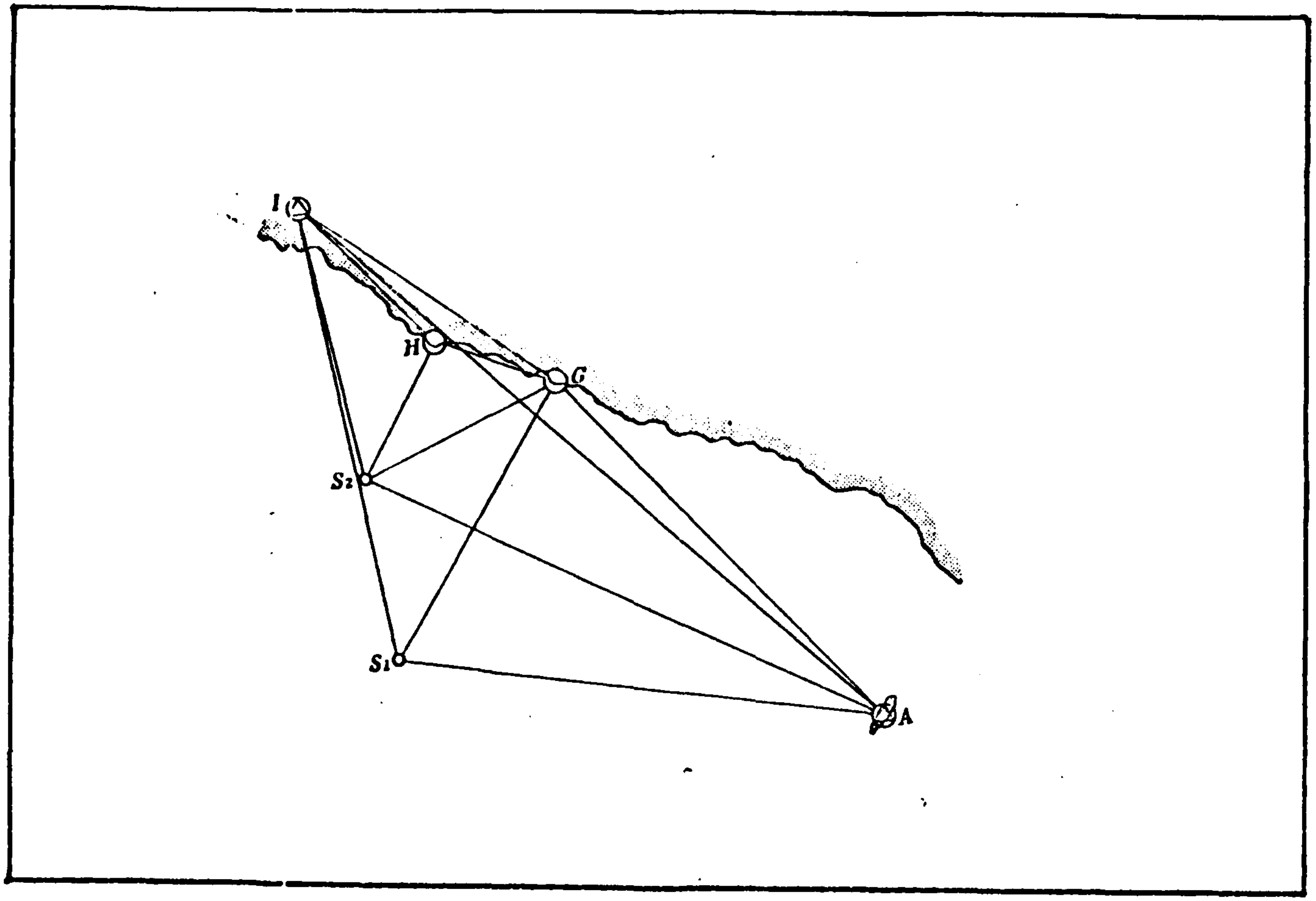

Figure 11. The Use of Temporary Floating Marks with Triangulation on an Open Coast [From the AMHS]

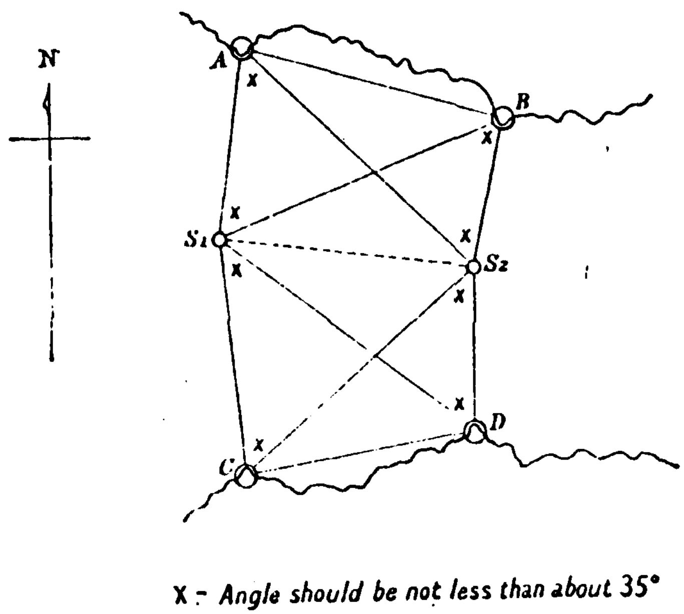

Figure 12. The Use of Temporary Floating Marks with Triangulation in Channels [From the AMHS]

ABS_2S_1 and CDS_2S_1 are strong figures. Simultaneous theodolite angles are observed from the four stations to the foremast for each position of the ship and stations S_1 and S_2 are established through the triangles ABS_1 and ABS_2 whose side AB is known and angles at A and B measured. Then after the establishment of the temporary stations S_1 and S_2, stations C and D are established through the quadrilateral S_1S_2DC in the following way. Side S_1S_2 is known and side CD is measured. Sides S_1C, S_2D and diagonals S_1D and S_2C are determined from the triangles S_1DC and S_2DC whose side DC and angles at D and C have been measured.

Another possibility is to use offshore moored beacons as temporary floating marks. This method is illustrated on Figure 13 where stations A and B already exist and horizontal control has to be established between B and F. Four beacons are moored offshore to form a strong triangulation net of adjacent quadrilaterals. The beacons are placed approximately opposite each shore station, such that ideally the quadrilaterals (BCba, CDcb, and so forth) are squares. Three observers are necessary to occupy stations A, B and C and measure simultaneous theodolite angles to beacon a. Station C can be established via the quadrilateral ABCa, and the procedure is repeated with the occupation of stations BCD for the establishment of station

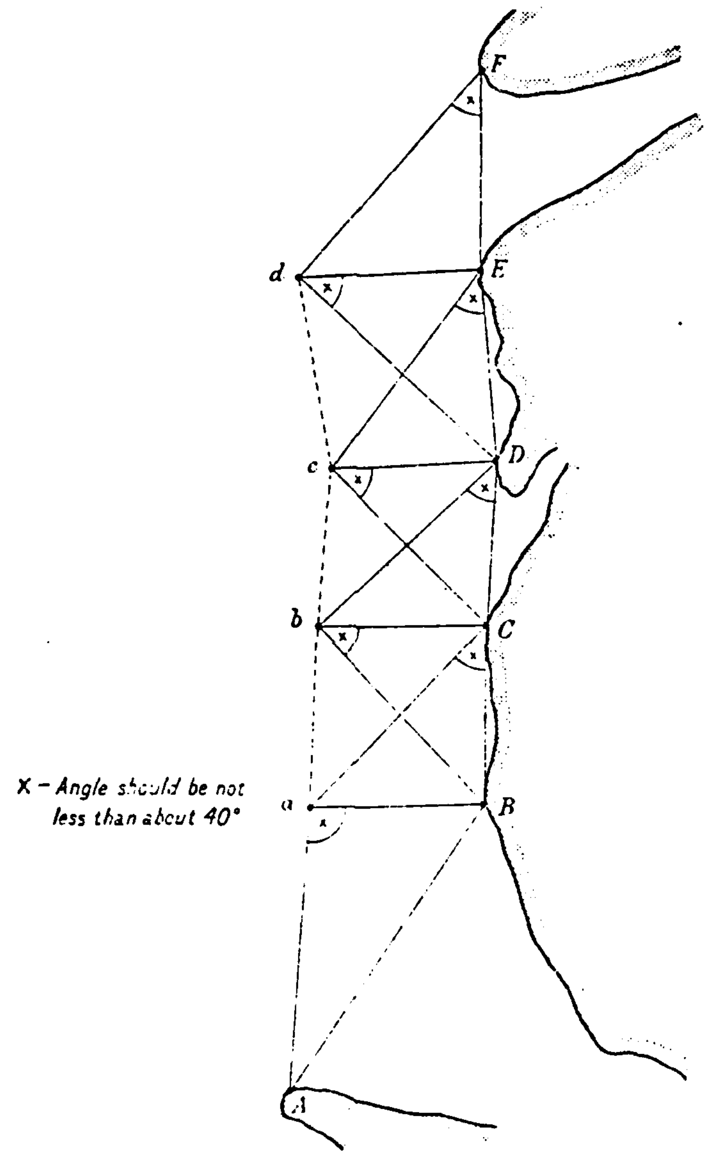

Figure 13. Triangulation with the Use of Off-Shore Beacons [From the AMHS]

D and so on up to station F. For better results, the observations to each beacon should be taken two or three times and the calculation of each side for each set of observations done independently. The results are finally meaned.

b. Triangulation Afloat and Floating Beacon Traverse

These methods are used when surveying with visual sextant methods are at such a distance from the land that the onshore signals cannot be clearly seen. Floating offshore stations in the form of anchored beacons are utilized to extend the control offshore. The difference between these methods and the previously described use of temporary floating marks with triangulation is that triangulation afloat and floating beacon traverse methods are used to extend the horizontal control offshore while the use of temporary floating marks with triangulation is used in order to establish horizontal control ashore. Unlike the use of temporary floating marks, triangulation afloat and floating beacon traverse induce large errors due to the movement of the beacons around their anchors. In order to minimize these errors, the anchor lines should have a short scope and the lines from shore should be as long as possible (seven to eight mile lines of sight can be observed under good conditions). The use of these methods would be precluded by use of an electronic positioning system.

E. THE HELLENIC NAVY HYDROGRAPHIC SERVICE METHODS AND PROCEDURES FOR ESTABLISHMENT OF HORIZONTAL CONTROL

In Greece, as in Great Britain, the agency responsible for the establishment and maintenance of the national geodetic network, is independent from the agency responsible for the hydrography of the country's waters. For the establishment of hydrographic horizontal control, the Hellenic Navy Hydrographic Service maintains and expands a hydrographic horizontal (and vertical) control network. The stations in this network in most cases are established by direct connection with one of the higher accuracy national horizontal control networks which are maintained by the Hellenic Army Geodetic Service. The accuracy of the above hydrographic horizontal control network is 1 part in 10,000 (equivalent to the U.S. NOS Third Order Class I accuracy). For secondary stations which will not be used for the extension of control, lower accuracies are permitted.

The methods used for the establishment of horizontal control are mainly triangulation and secondly, traverse. The fact that triangulation is the most popular method in the HNHS while in the other countries already examined the preferred method is traverse, is attributed to the peculiarity of the Greek coasts. Greece is both a continental as well as an insular country. Although its size is relatively small (about half the size of California), the developable length of its coasts is about

15,500 kms which is about the length of the coasts of the African continent. Numerous peninsulas, gulfs, bays and harbors are formed in the small area of the continental country, while the number of islands, islets and larger uncovered rocks at distances greater than 200 m from the coast number about 3100. The above peculiar geographic configuration is ideal for triangulation methods, particularly resection[12] and/or intersection[13].

Traverse methods are generally used for coastlining. The observational procedures and standards for triangulation or traverse surveys to densify the hydrographic horizontal control network are identical to the British ones used for regular triangulation surveys and accurate traverses. Secondary stations which will not be used to further extend the control (like T-2 theodolite stations from which the survey vessel is positioned) are usually located by minor traverses.

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[12]Resection: A graphical or analytical determination of position as the intersection of at least three lines of known relative direction to corresponding points of known position [Ref. 35].

[13]Intersection: The procedure of determining the horizontal position of an unoccupied point by direction observations from two or more known positions [Ref. 36].

III. TIDES AND DIFFERENTIAL LEVELING

The hydrographic surveyor, having established his horizontal control, is able to relate the position of his vessel to this reference system (horizontal control) with various positioning methods which will be discussed in the next section. To start the hydrographic operations, he needs a vertical reference plane to which depths will be referenced -- a sounding datum. The sea surface cannot be used as a sounding datum because it is not fixed, but is subject to vertical fluctuations due to wind and tides. A sounding datum (like mean lower low water (MLLW)) is referred to some phase of the tide and is usually related to a number of defined physical reference marks or benchmarks[14] so that it can be easily recovered during any future survey. Sounding datums should not be confused with chart datums which are those to which the depths of the final published chart are reduced. Although the coincidence of sounding and chart datums greatly facilitates the further charting processes, it is not an absolute requirement. A sounding datum may be established and be different from the chart datum so that depths can be measured and reduced to it

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[14]Benchmark: A permanent, stable object containing a marked point of known elevation.

and at a later time converted to the appropriate chart datum. This is particularly true when the chart datum is difficult to establish or has not been established and tied to existing physical marks or benchmarks during a previous hydrographic survey. Different chart datums are used by different countries, MLLW is a common one used in U.S. and Greece. Lowest Astronomical Tide[15] is the main chart datum used in Great Britain.

For the establishment of a sounding datum, a series of tidal observations in the area to be surveyed is required. According to the IHO S.P. 44 [Ref. 38]:

> "Tidal heights should be observed with an accuracy of at least 0.1 meter. Care should be taken that tidal observations are obtained for each of the tidal regimes which may occur within the area being sounded."

Many different methods exist for the establishment of a tidal sounding datum depending on the available tidal observations, the character of the tide (diurnal, semidiurral) and the proximity of the area in which the datum is to be established from the place where tidal observations are obtained. Such methods are explicitly described in special publications like the Admiralty Tidal Handbook No. 2 Datums for Hydrographic Surveys and the U.S. Coast and Geodetic Survey Special Publication No. 135 Tidal Datum Planes.

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[15] Lowest Astronomical Tide is the lowest level which can be predicted to occur under average meteorological conditions and under any combination of astronomical conditions [Ref. 37].

Once the sounding datum has been established, it is usually connected with any existing vertical control[16], or with some specially established benchmarks so that it can be easily recovered during future surveys. This connection is accomplished by the determination of the elevation difference between the tidal staff and the nearest benchmarks in the survey area. The method used for the determination of the above elevation difference is called differential leveling. In this method the height difference between two points A and B (Figure 14) are measured directly by means of a leveling instrument, and vertical leveling rods. The difference in reading between the two rods gives the elevation difference between points A and B. The major source of error in differential leveling is the "collimation error" which is the angle by which the line of sight of a leveling instrument deviates from the horizontal. This error can be minimized by making adjustments to the leveling instrument and by adopting appropriate measuring procedures such as balancing the foresight and backsight and limiting the sighting distances for each setup.

This section examines the specifications and procedures for the connection of a sounding datum with the vertical control used by the U.S. NOS, the Canadian Hydrographic

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[16] Vertical Control: A system of reference points used for the determination of vertical datums (planes) from which heights and depths are measured.

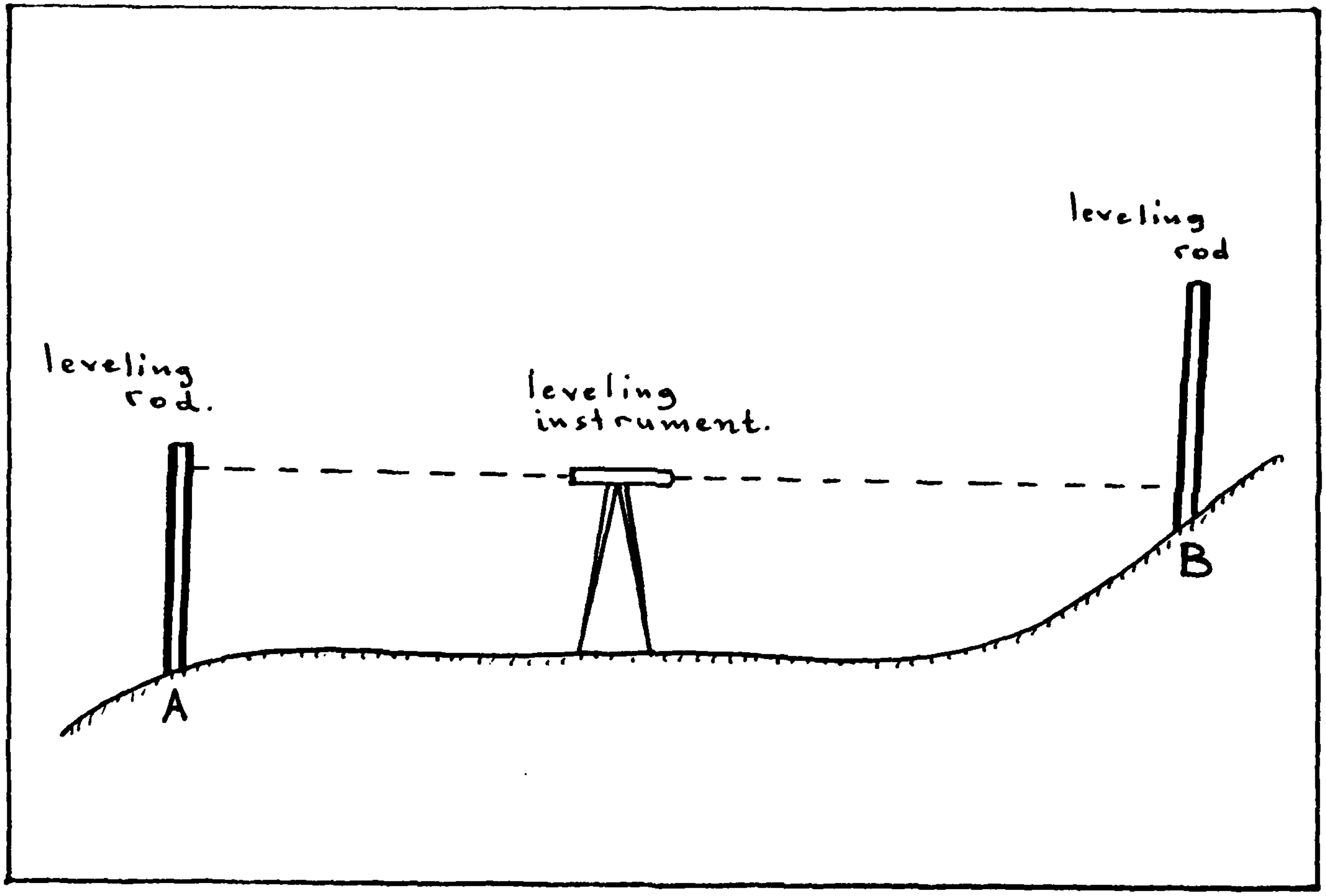

Figure 14. Differential Leveling

Service and the British Hydrographic Office. The Hellenic Navy Hydrographic Service does not have any specific standards but it follows those recommended by the AMHS and the GIHS. The IHO S.P. 44 also does not include any specifications on differential leveling.

A. THE U.S. NATIONAL OCEAN SURVEY METHODS

In the United States, vertical control is classified as first, second, third and lower order according to the degree with which error magnitudes are limited. In leveling, errors propagate as the square root of the distance surveyed. Table V shows the classification as well as the vertical control network characteristics. Each line of a vertical control network is divided into sections which connect two permanent control points (bench marks) and consist of an unbroken series of setups like that of Figure 14.

In hydrographic surveys "... for each continuous recording tide station or water level reference gage, five recoverable bench marks shall be established within a distance of 1 mile. Each of the bench marks must be connected to the gage staff (or measuring mark) by third order levelling" [Ref. 39]. For third order leveling, the NOS specifications require a maximum sighting distance of 90.0 m with maximum allowable imbalance per setup of 10.0 m. The sections should be double run, that is, from the tide

TABLE V

U.S. NOS VERTICAL CONTROL CLASSIFICATION

Classification	First Order		Second Order		Third Order
	Class I	Class II	Class I	Class II	
Relative accuracy between directly connected points or bench marks (standard error)	$0.5\text{mm}\sqrt{K}$	$0.7\text{mm}\sqrt{K}$	$1.0\text{mm}\sqrt{K}$	$1.3\text{mm}\sqrt{K}$	$2.0\text{mm}\sqrt{K}$
	(K is the distance in kilometers between points)				
Network Components	Basic Vertical Network A (Control Establishes the National Network)	Basic Vertical Network B	Secondary Vertical Network (Control develops the National Network)	Supplemental Vertical Network (Control contributes to the National Network)	Local Vertical Control
Recommended density of lines	100-300km	50-100km	25-50km	10-25km	As needed

staff to each bench mark and then back along the same path. The maximum allowable closing error between the forward and backward running of a section is 9.0 mm $\sqrt{K}$, where K is the length of the sections in kilometers. The maximum collimation error for a single line of sight should not exceed ±10.0" or 0.05 mm/m.

Two methods are used for collimation error check and adjustment [Ref. 40]. One is Kukkamakis method and the other is the 10-40 method. In both methods the collimation error is computed and if it exceeds its maximum allowable value (0.05 mm/m) the instrument is adjusted with the appropriate screws. The above procedure is repeated until the measured collimation error becomes less than 0.05 mm/m. The computation of the collimation error in both methods is performed through two different, but distinct setups made on flatest possible ground. In Kukkamakis' method the leveling rods are placed 20 meters apart, the leveling instrument is set up exactly at the middle of this distance and the rods are observed. The level is then moved to a point 20 meters beyond either of the two rods and again they are observed. In the 10-40 method, the distance between the leveling rods is exactly 50 meters. At the first setup the leveling instrument is positioned at 10 meter sighting distance from the foresight rod and 40 meter sighting distance from the backsight rod, while at the second setup the same instrument is 40 meters from the foresight rod and 10 meters from the backsight rod. In both methods, the collimation error is given by the formula:

$$C = \frac{[(\Delta h_1 - e_1) - (\Delta h_2 - e_2)]}{\Delta s_1 - \Delta s_2} \ \frac{mm}{m} \qquad \text{(III-1)}$$

where: Δh_1 and Δh_2 are observed elevation differences (in mm) for each setup.

e_1 and e_2 are curvature and refraction corrections (in mm) for each setup taken from Table VI.

Δs_1 and Δs_2 are the imbalances (in meters) in each setup (difference between foresight and backsight distances).

For Kukkamakis' method, $\Delta s_1 = 0$ and $e_1 = 0$, so formula III-1 becomes:

$$C = \frac{[\Delta h_1 - (\Delta h_2 - e_2)]}{-\Delta s_2} \qquad \text{(III-2)}$$

B. CANADIAN HYDROGRAPHIC SERVICE METHODS

In Canada, vertical control is classified as first, second, third and fourth order according to the allowable discrepancy between independent forward and backward levelings between bench marks. In hydrographic surveys, fourth order differential leveling is used [Ref. 41]. According to the Canadian classification the maximum allowable discrepancy between independent forward and backward levelings for fourth order is ± 120 mm K

TABLE VI

REFRACTION AND CURVATURE ERRORS IN A SINGLE SIGHT

Sighting distance, s (m)	Sighting distance, s (ft)	Error in a rod reading, e (mm)	Error in a rod reading, e (ft)
0 to 28	0 to 92	0.0	0.000
28 48	92 157	0.1	0.000
48 61	157 200	0.2	0.001
61 73	200 240	0.3	0.001
73 82	240 269	0.4	0.001
82 91	269 299	0.5	0.002
91 99	299 325	0.6	0.002
99 106	325 348	0.7	0.002
106 113	348 371	0.8	0.003
113 119	371 390	0.9	0.003
119 125	390 410	1.0	0.003
125 131	410 430	1.1	0.004
131 137	430 449	1.2	0.004
137 142	449 466	1.3	0.004
142 147	466 482	1.4	0.005
147 150	482 492	1.5	0.005
160	525	1.8	0.006
170	558	2.1	0.007
180	591	2.3	0.008
190	623	2.6	0.009
200	656	2.8	0.009
210	689	3.0	0.010
220	722	3.3	0.011
230	755	3.7	0.012
240	787	4.0	0.013
250	820	4.3	0.014
260	853	4.7	0.015
270	886	5.0	0.016
280	919	5.4	0.018
290	951	5.8	0.019
300	984	6.2	0.020

[From the NOAA Manual NOS NGS 3, Geodetic Leveling, 1981] where K is the distance between benchmarks in kilometers measured along the leveling route. Provided that the discrepancy between the forward and backward runnings is within the above tolerances, the difference in elevation is the mean of the two runnings.

As it is stated in the Canadian specifications [Ref. 42]:

> "It is preferable that the difference of elevation between successive bench marks be determined twice by two independent levelings." And, "..., it is extremely desirable to use the two-rod system and to keep balanced foresights and backsights."

No other specifications or procedures for fourth order differential levelling and collimation error check and adjustment are mentioned in the Canadian specifications or standing orders.

C. BRITISH HYDROGRAPHIC DEPARTMENT METHODS

The requirement of the British Hydrographic Department for vertical control in hydrography are stated in the GIHS [Ref. 43]. "Sounding datum must always be connected to at least two fixed marks on shore, and where there is a land leveling system available, connection to this must also be made ... Levels are always to be given to two decimal places of a meter."

In addition to the above statement, the AMHS suggests the following observational procedures in order to eliminate errors in differential leveling.

(1) Balance the lengths of foresights and backsights either by pacing or by tacheometric methods in greater distances.

(2) Design the setups so that no line of sight is allowed to pass within a foot of the ground.

(3) Observe foresights and backsights as quickly as possible.

(4) Hold the leveling staff within a degree of the vertical. To achieve this, use the level bubble or sway the staff gently backwards and forwards in the plane of the line of sight, taking the smallest reading as the correct one.

(5) Run the leveling distance twice to check for errors.

(6) Check and adjust the leveling instrument for collimation error.

In addition to the above rules, the AMHS provides the following tables (Tables VII and VIII) showing the maximum allowable discrepancies between the two levelings of the line (Case 5 of the above rules).

For collimation error check and adjustment, the AMHS suggests the following simple and quick method. Two sheets of thick paper (Figure 15) are fixed on the walls of a building at C and D so that the lines of sight from the level fall on them. The distance CD should be 100 to 150 feet and other firm objects like telegraph poles or trees can also support the two paper sheets. The instrument is then levelled at point A so that distances AC and AD are equal to within a foot. The boards C and D are shot and

TABLE VII

ERROR IN MEAN DIFFERENCE OF LEVELS NOT TO EXCEED 0.1 FEET
(ADMIRALTY MANUAL)

LENGTH OF LINE OF LEVELS NOT MORE THAN	MAXIMUM LENGTH OF SIGHT	BACK AND FORESIGHTS TO BE EQUAL TO WITHIN	ALLOWABLE DISCREPANCY BETWEEN TWO LEVELLINGS
4 cables*	400 feet	30 feet	0.2 feet
15 miles	250 feet	10 feet	0.25 feet

*The value of one cable in the British Navy is 608 feet or exactly one-tenth of a nautical mile. In the U.S. Navy it is 720 feet, but it is infrequently used [Ref. 44].

TABLE VIII

ERROR IN MEAN DIFFERENCE OF LEVELS NOT TO EXCEED 0.01 FEET
(ADMIRALITY MANUAL)

LENGTH OF LINE OF LEVELS NOT MORE THAN	MAXIMUM LENGTH OF SIGHT	BACK AND FORESIGHTS TO BE EQUAL TO WITHIN	ALLOWABLE DISCREPANCY BETWEEN TWO LEVELLINGS
1 cable*	-	30 feet	0.01 feet
5 cables*	250 feet	10 feet	0.02 feet
2 miles	150 feet	3 feet	0.03 feet

*The value of one cable in the British Navy is 608 feet or exactly one-tenth of a nautical mile. In the U.S. Navy it is 720 feet, but it is infrequently used [Ref. 44].

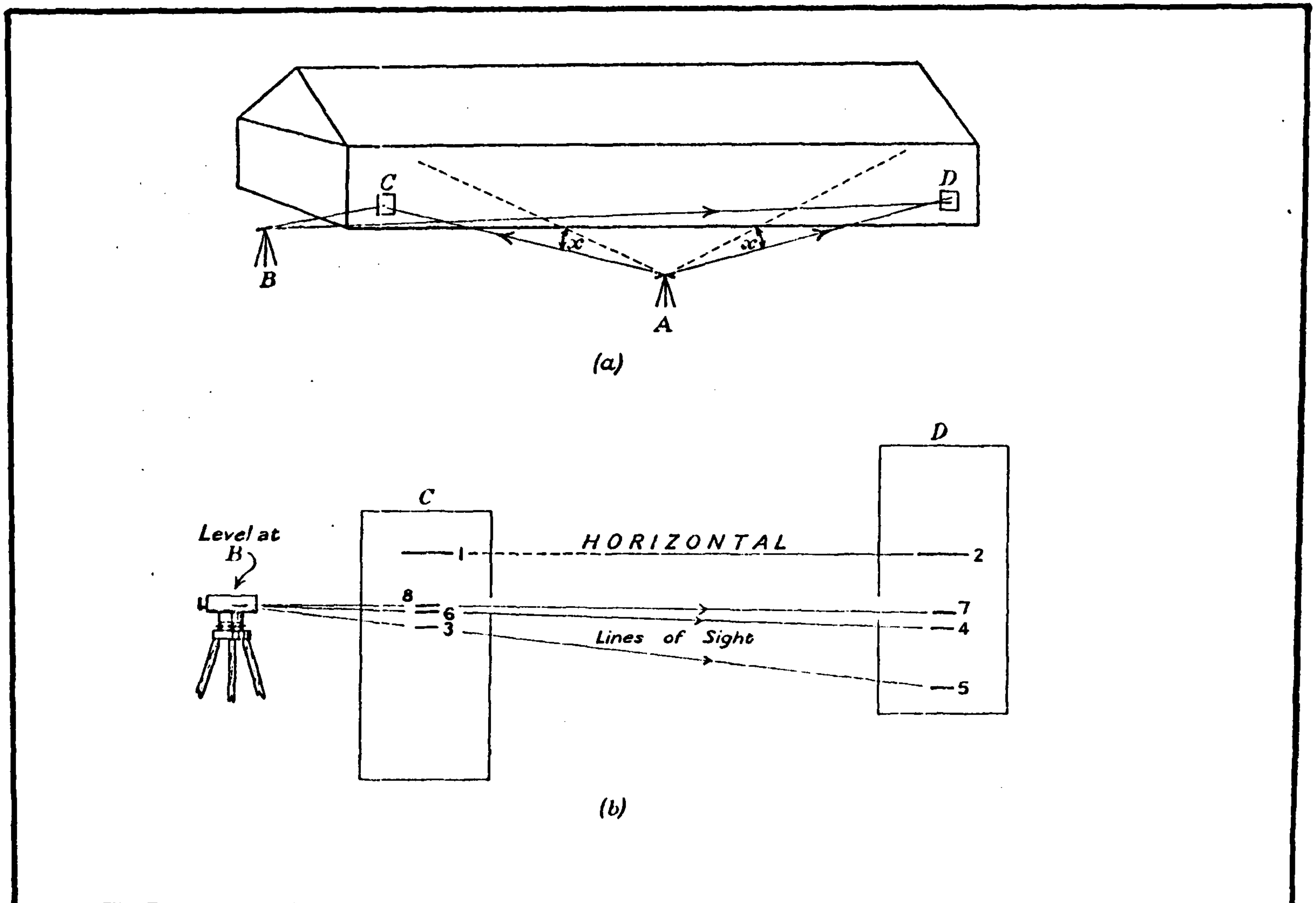

Figure 15. Leveling Instruments Collimation Error Check and Adjustment [From the AMHS]

marks 1 and 2 are drawn with a sharp pencil to show the level where the lines of sight cut the boards. Marks 1 and 2 lie on the same level and so they define a datum which will be used for the check and adjustment.

The leveling instrument is then set at point B so that the distance BC is much less than BD. The procedure is greatly facilitated if BD is at least 10 x BC. With the instrument leveled so that the level of its line of sight approximates the established datum 1-2, the board A is shot and mark 3 is drawn at the intersection of the new line of sight with the board. The vertical distance 1-3 is measured on board C and then mark 4 is drawn on board D so that distance 2-4 is equal to 1-3. Now board D is shot again and mark 5 is drawn at the intersection with the line of sight. If no collimation error exists, marks 5 and 4 must coincide, otherwise the optical axis is adjusted so that the intersection of the line of sight falls exactly on mark 4. Now the adjusted instrument is checked again by shooting board C. If mark 6, showing the intersection of the adjusted line of sight with board C, coincides with mark 3 the instrument is properly adjusted, otherwise another adjustment is necessary. Usually two adjustments are adequate but large collimation errors may need more.

IV. HYDROGRAPHIC SPECIFICATIONS

As it is stated in the introduction of the IHO S.P. 44 [Ref. 45]:

> "The planning for each hydrographic survey and the preparation of appropriate specifications is a unique task, and it is not possible to prepare a treatise on accuracy standards for hydrographic surveys which would be applicable for any area to be surveyed. The density of soundings and the precision of measurements depends on several factors: the depth of water, the composition and configuration of the bottom, and the draft of ships which will navigate in the area all need to be considered."
>
> "Certain degrees of accuracy are nevertheless, commonly acceptable for hydrographic operations, and it is reasonable that such standards should be stated in order that they may serve as a guide for planning an adequate hydrographic survey."

This section examines the hydrographic standards recommended by the IHO and those actually employed by the four considered agencies.

A. SCALE OF THE SURVEY

The IHO recommendations start with the scale of the survey. The IHO guidelines for the selection of the scale of the survey are summarized in Table IX.

Different agencies adopt different standard scales on which their surveys are conducted. The U.S. NOS has adopted a basic scale of 1:20000. Almost all other survey scales have a simple ratio to this basic scale but scales of 1:30000 or 1:50000 can be occasionally used. For surveys of

TABLE IX

SCALES OF SURVEY RECOMMENDED BY IHO

Survey type:	Scale
Ports, harbors, channels, pilotage waters.	1/10000 or larger
Harbor approaches and other waters used frequently by shipping.	1/20000 or larger
Coastal areas to a general depth of at least 30 meters (40 meters where super-deep-draught vessels are expected to operate, or where the existence of wrecks or other hazards are suspected).	1/50000 or larger
Hydrographic surveys in depths between 30 meters and 200 meters (under special circumstances depending on many factors, the most crucial of which are the importance of the area covered and the depth and bottom configuration).	1/50000 - 1/100000

important harbors and anchorages, scales of 1:10000 or larger are used. Larger scales used by the U.S. NOS are 1:5000 and 1:2500 as well as multiples of 1:1000. The British Hydrographic Department, the Canadian Hydrographic Service and the Hellenic Navy Hydrographic Service all have adopted a basic scale of 1:25000. Larger scales usually used by the British Hydrographic Department and the Canadian Hydrographic Service are 1:15000, 1:12500, 1:8000, 1:4000, and 1:2000.

The HNHS usually performs larger scale surveys at 1:10000, 1:5000 and 1:2000. For large scale surveys of pier and docks the British Hydrographic Department and the Hellenic Hydrographic Service usually use two basic scales: 1:200 and 1:1000 while the Canadian Hydrographic Service uses the scales 1:600 and 1:1200.

B. INTERVAL BETWEEN SOUNDING LINES

For the spacing of sounding lines, the IHO recommends a maximum permissible interval between principal sounding lines of no more than 10 mm at the scale of the survey. For cross check sounding lines, an amount of no more than 10% of the principal sounding lines is recommended (by IHO). As shown in Table X, there is a tendency of some agencies to adopt a closer sounding line interval.

TABLE X

RECOMMENDED SPACING OF SOUNDING LINES

Recommended by	Principal Lines	Cross Lines
IHO	not more than 10 mm	normally not more than 10%
U.S. NOS	not more than 10 mm	between 8 and 10%
British Hydrographic Department	5 mm at least up to 50 m depth*	not specified
Canadian Hydrographic Service	6 mm up to 37 m (20 fms) depth	14% for depths <183 m (100 fms)
	10 mm for depths > 37 m (20 fms)	7% for depths >183 m (100 fms)
Hellenic Navy Hydrographic Service	5 to 8 mm	Between 5 and 10%

* "... When the bottom is very regular with sand or mud, in depths of over 50 meters, or in very shallow water where navigation will be confined to boats, lines of soundings may be opened out ..." [Ref. 46]

Some special standards are required for certain situations by some agencies. The U.S. NOS provides the following detailed specifications [Ref. 47]:

Maximum allowable spacing: 1.0 cm

Harbors and restricted areas:	
depth <20 fm	spacing 100 m
20-30 fm	200 m
>30 fm	400 m
Dredged or natural narrow channels	50 m
Survey scale is 1:5000 or larger	50 m

Open Coast:

Regular, smooth bottom

depth <20 fm	spacing 200 m
20-30 fm	400 m
30-110 fm	800 m

Entrance to harbors and areas adjacent to spits or rocky points, reduce spacing by half.

Irregular bottom

Rocky points, spits, entrances with

depth <20 fm	spacing 100 m

Other

depth <20 fm	spacing 200 m
20-30 fm	400 m
30-110 fm	800

The Hellenic Navy Hydrographic Service provides the following specifications for survey of small bays and anchorages conducted at 1:2000 scale which are included specified in project instructions:

> Sounding lines should be determined via transits (or visual ranges) established on the coast and include skiff and launch sounding lines as well as crosslines. The skiff sounding lines are spaced 10 meters apart (5 mm at the scale of survey). The hydrographic launch sounding lines should be on the extension of the skiff's sounding lines (on the same transits) spaced every 10 or 20 meters and should have an overlap zone of at least 10 meters with the area surveyed by the skiff. Crosslines should be run perpendicular to the principal sounding lines spaced about every 60 meters.

C. SPACING OF POSITION FIXES AND SOUNDINGS

Table XI depicts the various specifications concerning the spacing of position fixes along a sounding line and the interval between intermediary soundings (those plotted between successive fixes along a sounding line). The CHS and the HNHS have established some detailed specifications for large scale surveys of piers, docks and wharves. For

TABLE XI

RECOMMENDED SPACING OF FIXES AND INTERMEDIARY SOUNDINGS

Recommended by	Fix Interval Along a Sounding Line	Interval of Intermediary Soundings
IHO	not more than 40 mm	not more than 10 mm except for even seabed when it can be increased to 10 mm
U.S. NOS	not more than 50 mm when a position is determined and recorded for each sounding. In other cases not more than 40 mm. For visually controlled surveys when not streered on lane or are not more than 35 mm.	between 4 and 6 mm
British Hydrographic Department	10 to 25 mm	2.5 mm
Canadian Hydrographic Service	20 - 40 mm	not more than 6 mm except in areas of even bottom where it can be increased
Hellenic Navy Hydrographic Service	not more than 40 mm	between 4 and 6 mm

large scales at wharves, the CHS's "Standing Orders" recommend the following [Ref. 48]:

> "... The scales normally used for wharf plans are 100 feet to the inch (1:1200), or 50 feet to the inch (1:600) and this will usually be noted in the project assignments."
>
> Soundings close to wharf -- the spacing of the first three soundings is to be 6 feet, 12 1/2 feet and 25 feet from the wharf. At 100 foot to the inch (1:1200), the sounding 6 feet off cannot be shown without crowding, but it need not be inked on the plan unless the depth is shoaler than that of the next sounding out. In such cases, a note shall appear in the title indicating the distance of the first sounding from the wharf.
>
> Soundings farther off wharf -- the remaining soundings will normally be spaced at 25 foot intervals. However, this will depend to some extent on the depths encountered and also on the incidence of shoals in the area."

Piers and docks are surveyed by the HNHS at two scales, either 1:200 or 1:1000 as follows [Ref. 49]:

Scale of survey 1:200

(1) Sounding line spacing: 1 m (5 mm at scale of survey).
(2) Soundings taken with leadline.
  a. Every 1 m from 0-5 m from pier.
  b. Every 2 m from 6-19 m from pier.
  c. Every 5 m from 20-60 m from pier.
(3) Sweepings: Should be performed in two directions perpendicular and parallel to the pier. The depth of the sweep should be 1 to 2 meters deeper than the expected maximum vessel draft to use the pier.
  -- The type of sweeps used by the HNHS are of the pipe drag type, i.e., a bar held horizontal below and perpendicular to the launch's keel suspended by chain.

Scale of survey 1:1000

(1) Sounding line spacing: 5 m (5 mm at scale of survey).
(2) Soundings should be taken with leadline at 0 and 1 m from pier and then every 5 meters.

For both scales 1:200 and 1:1000, the following procedure should be followed for the selection of the depth which will be finally listed on the smooth sheet at the edge of the pier.

(1) The soundings at 0 and 1 m from pier should be tabulated.

(2) If the above values differ by more than 1 to 1.5 meters, a special report for the reasons of the difference is required.

(3) The final selection of the depth to be put at the edge of the pier should be done at the office based on the above specific field report.

D. MEASURED DEPTHS AND BOTTOM SAMPLING

For the required accuracy of the measured depths, the IHO S.P. 44 recommends some maximum permissible errors which are shown in Table XII.

TABLE XII

MAXIMUM ERROR IN DEPTH MEASUREMENTS RECOMMENDED BY IHO

Depth	Maximum Error
0 - 30 m	0.3 meter
30 m - 100 m	1.0 meter
greater than 100 m	1% of depth

For the reduction of measured depths, the IHO specifications require:

"Measured depths must be reduced to the sounding datum by application of the tidal height. The error of such reductions should not exceed the errors acceptable for depth measurement specified in Table XII. Depths greater than 200 m normally need not be reduced for tidal height." [Ref. 50]

The allowable discrepencies at the intersections of principal and crossing sounding lines, according to the IHO specifications, should not exceed twice the values of Table XII. Other standards for depth measurements which differ from those of the IHO are required by some agencies. The U.S. NOS requires that

> "Depth measuring instruments or methods used to sound over relatively even bottoms or in critical depths should measure depths less than 20 fm to within 0.5 ft accuracy -- greater depths to within 1% accuracy. In rapidly changing depths and over irregular bottoms, accuracy requirements may be decreased to 1 ft in depths less than 20 fm." [Ref. 51]

For the intersection of sounding lines the discrepancies acceptable by the U.S. NOS are:

> "In areas of smooth bottom with depths less than 20 fm, discrepancies should not exceed 2 ft or 0.4 fm. In areas of irregular bottom and in depths greater than 20 fm, discrepancies should not exceed 3% in the lesser depths and should not exceed 1% in ocean depths." [Ref. 52]

The accuracies for depth measurements required by the Canadian Hydrographic Service are:

0 - 20 m:	0.3 m
20 - 100 m:	should strive for 0.5 m
>100 m:	1% of depth

The maximum permissible discrepancies at intersections of sounding lines, according to the Canadian specifications, are 0.3 m for depths less than 10 m and 3% of the depth for depths greater than 10 meters.

The various specifications for the required density of bottom samples are as follows:

(1) IHO:

"Samples of the bottom should be obtained in depths less than 100 meters to provide information for anchoring. As a general guide, sampling of the bottom should be spaced as follows:

(a) In general, at intervals of 10 cm at the scale of the survey.

(b) In areas expected to be used as anchorages, as necessary to indicate the limits of different types of bottom." [Ref. 53]

(2) U.S. NOS:

"In anchorages, the distance between bottom samples should not exceed 5 cm at the scale of the survey. The distance between samples in other areas on inshore surveys should not exceed 6 cm. In depths less than 100 fm in offshore survey areas, the distance should not exceed 12 cm. For ocean surveys conducted between the 100 and 1000 fm depth contours, the character of the bottom is determined at intervals of about 8 to 16 km... In harbors and anchorages, enough information should be obtained to permit the delineation of the approximate limits of each type of bottom." [Ref. 54]

(3) British Hydrographic Department:

"Natures of the bottom are to be obtained at frequent intervals throughout the survey area. The accepted guide line is to obtain one 'bottom' sample to every 5 cm square on paper, at the scale of the survey." [Ref. 55]

(4) Canadian Hydrographic Service:

"In waters that may be used for anchoring, samples should be taken at regular intervals not to exceed 5 cm (2 in) at the scale of the survey. In other areas, shoaler or deeper, a spacing of 8 cm (3 in) is sufficient depending on the regularity of the bottom." [Ref. 56]

(5) Hellenic Navy Hydrographic Service

"Bottom samples should be taken every 7.5 cm at the scale of the survey for depths up to 50 meters, and every 10 cm for depths between 50 and 100 m." [Ref. 57]

E. POSITIONS

The minimum required position accuracy of soundings, dangers and all other significant features recommended by the IHO should be such that:

> "... any probable error, measured relative to shore control, shall seldom exceed twice the minimum plottable error at the scale of the survey (normally 1.0 mm on paper)."

This statement, rather than presenting minimum requirements for position accuracy, is very ambiguous and subjective. As previously mentioned, probable error is associated with 50% probability. The phrase "shall seldom exceed" has been interpreted by Munson [Ref. 60] to mean 90% probability. "Minimum plottable error" is even more subjective, although it would appear to mean the minimum plotting error that can be detected by the human eye. If this definition were correct, then the plotting material would be irrelevant. A suggested rewording of the IHO statement for positioning accuracy is that for any position the probable error shall not exceed 1.0 mm at the scale of the survey.

Of particular interest and value is the method adopted by the U.S. NOS using the root mean square error (rmse) or drms to estimate position accuracies. The drms is based on the standard errors for each one of two lines of position used to determine a fix. It represents the radius of a circle containing approximately 65% of the plotted fixes. The determination of the rmse is done via the formula

$$d_{rms} = \sqrt{\sigma_1^2 + \sigma_2^2} \csc\beta$$

where: σ_1, σ_2 are standard errors of position lines 1 and 2 in distance units, and β is the angle of intersection between the lines of position at the vessel.

Other expressions for drms are given in the NOS Hydrographic Manual as well as in special studies like those of Heinzen [Ref. 58] and Kaplan [Ref. 59]. The U.S. NOS Hydrographic Manual provides some specifications for positional accuracy when range-range electronic positioning systems are used. These specifications require that hyperbolic and phase comparison systems operating in a range-range mode should not be used in areas where the rmse exceeds 0.5 mm at the scale of the survey.

> "... Super high frequency direct distance measuring systems shall be used for hydrographic positioning control only ... where ... the following conditions are met:
>
> drms ≤
> - 0.5 mm at the scale of the survey for scales of 1:20000 and smaller.
> - 1.0 mm at the scale of the survey for 1:10000 scales surveys.
> - 1.5 mm at the scale of the survey for scales of 1:5000 and larger." [Ref. 61]

Other U.S. NOS specifications required for positional accuracy concern the accuracy of horizontal angles when range/azimuth or visual methods are used to locate the vessel. In the case of a range azimuth positioning method, the NOS Hydrographic Manual states that

> "... objects sighted on for initial azimuths should be at least 500 m from the theodolite. ... Observed azimuths or directions to the sounding vessel for a position fix shall be read to the nearest 1 min of arc or better if necessary to produce a positional accuracy of 0.5 mm at the scale of the survey." [Ref. 62]

For T-2 theodolite intersections, if angles are observed to 1 min of arc and the angle of intersection at the vessel is between 30° and 150° then the resulting positional error will be no more than 1.0 mm at the scale of the survey [Ref. 63]. As far as sextant three-point-fix accuracy is concerned, the NOS Hydrographic Manual provides some useful positional error contours for various configurations of the three-point-fix which are presented in the next section on positioning methods. No similar specifications or requirements for position accuracy could be found for the other agencies considered in this study.

V. HYDROGRAPHIC METHODS AND TECHNIQUES

The specifications presented in the previous section do not ensure that all the required minimum accuracy standards can automatically be met by simply following the few stated simple rules. In order to conduct an efficient survey, the individual hydrographer is called upon to use his experience, common sense, knowledge and often his imagination. He not only has to choose between different methods, but may also be called to modify existing ones and sometimes even to invent others. The following methods and combinations of methods are some of the possible ways available to the surveyor to achieve his goal.

A. POSITIONING METHODS

1. Sextant - Three-Point Fix

One of the oldest and historically most widely used methods of fix determination for hydrographic surveying is the three-point sextant fix. The concept of the method, illustrated in Figure 16, is very simple. Two horizontal sextant angles θ_1 and θ_2 are observed simultaneously between three known points A, B and C. The vessel's position P is then determined via resection computation at the intersection of the three lines of position (LOP). One LOP is the circle defined by the known points A and B and

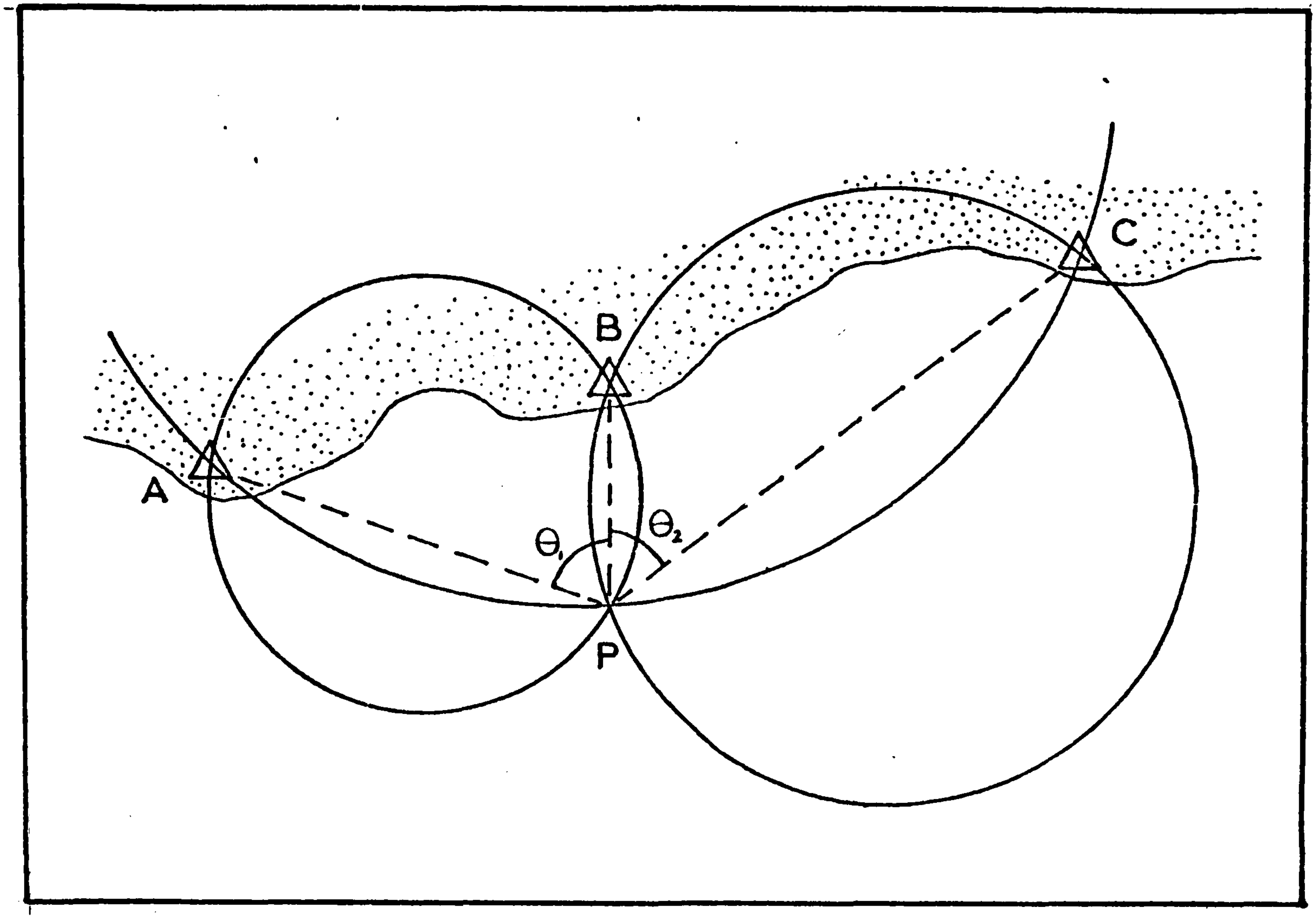

Figure 16. The Three-Point Fix

the angle θ_1. The second LOP in the circle defined by the known points B and C and the angle θ_2, while the last LOP is the circle defined by points A and C and the angle $\theta_1 + \theta_2$. In reality, only two LOPs are determined because only the angles θ_1 and θ_2 are observed while angle $\theta_1 + \theta_2$ is inferred not measured. The fix is easily plotted by a three-arm protractor. Of particular interest is the use by the U.S. NOS of the electronic digital sextant which has been specially designed to provide accurate angular data to the HYDROPLOT automation system of the NOS [Ref. 64]. The instrument is used in a manner similar to that of a conventional sextant except that angles are not read, but they are automatically recorded in order to provide machine plotted positions.

The accuracy of the three-point fix has been thoroughly examined in specific studies but is not easily quantified. The NOS Hydrographic Manual is one of the numerous sources where potential errors in the three-point-fix are examined. A more detailed analysis of potential errors in the three-point fix is presented by Mills [Ref. 65].

Figure 17, taken from NOS Hydrographic Manual, can be used to estimate the positional accuracy of various configurations of the three-point fix. The error contours correspond to errors of 1 minute of arc in each observed angle and to a horizontal control accuracy of 1:10,000.

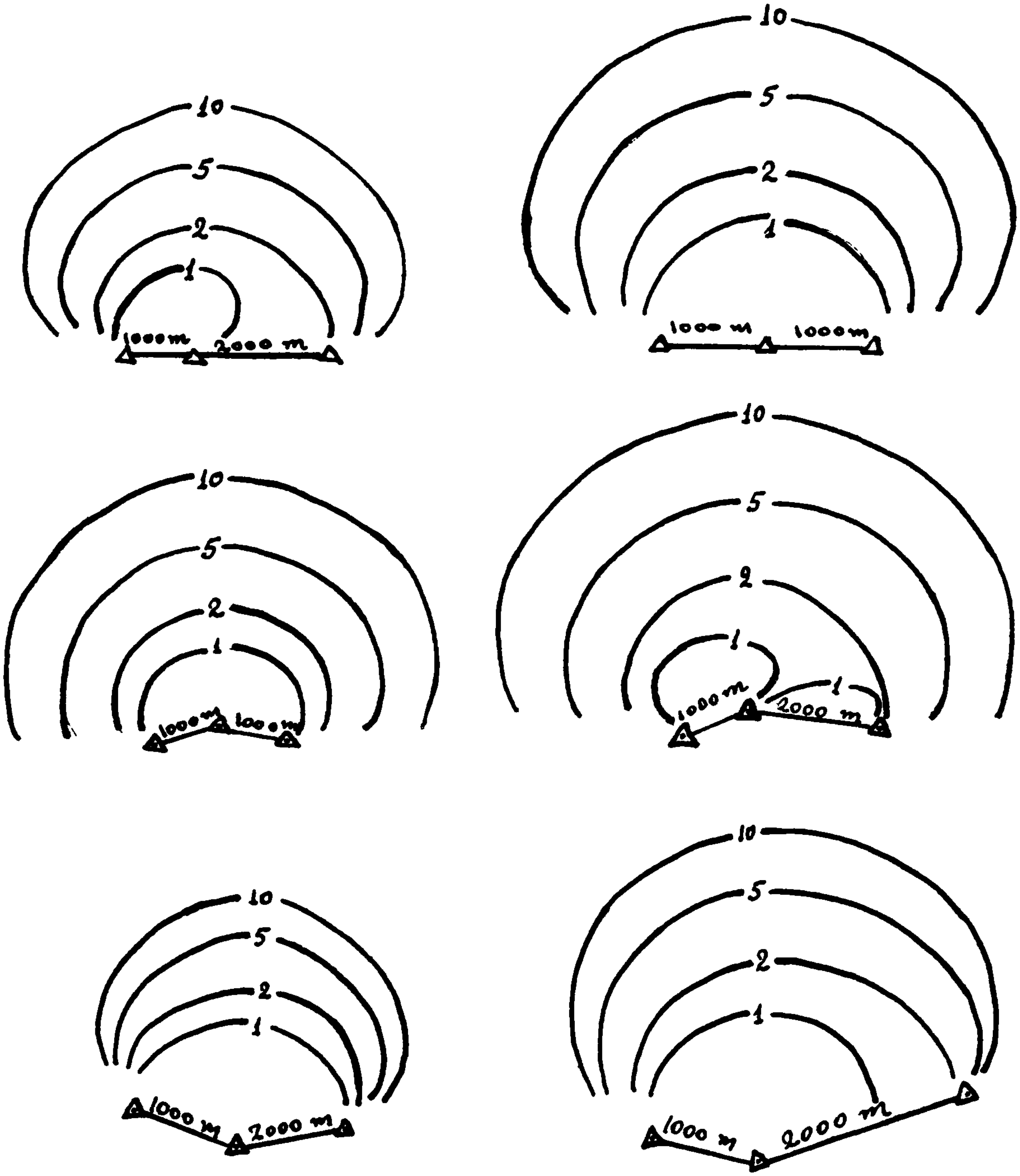

Figure 17. Error Contours in Meters for Various Configurations of the Three-Point Fix [From the NOS Hydrographic Manual]

Since the relationship between observational errors and error contours is almost linear, the contours can be used for the estimation of positional errors corresponding to other than 1 minute of arc error in each observed angle. For example, for a 2 minute of arc error in each observed angle, the contour of 1 meter positional error will now represent a 2 meter positional error.

The introduction of electronic positioning systems has made this positioning method almost obsolete. The U.S. NOS estimates that the percentage of its surveys conducted by three-point fix is less than 1%, while the HNHS does not use this method any more.

The major advantage of the sextant three-point fix method is the simplicity and low cost of the equipment required and its major disadvantages are its dependence on the visibility over the surveyed area, the construction of many signals ashore, and the many potential errors associated with the method.

2. Electronic Positioning Systems (EPS)

Although much detailed analysis of electronic positioning systems (EPS) is available in various texts, papers and reports like Laurila (1975) and Munson (1977), a summarized overview is presented because EPS are the most common positioning methods in hydrographic surveying. The use of EPS in hydrography has greatly changed the way in which traditional hydrography was done. The tedious and time consuming operation of establishing a large number of signals required for visual methods is unnecessary when EPS are used. In general, electronic positioning systems bear the following advantages when compared with other conventional methods.

(1) Long range ability.

(2) High accuracy of measurement, particularly at long range.

(3) Ability to function in poor visibility.

(4) Instantaneous and continuous fixing operation.

(5) Ability to follow exact tracks (sounding lines) along a circular or hyperbolic arc.

(6) Automation capability.

The major disadvantages of EPS are the high cost of equipment and the requirement for highly trained maintenance personnel. A tremendous number of different positioning systems exist, but they can be generally classified in two ways -- according to fix geometry or measurement principle. Fix geometry refers to the way in which lines-of-position are determined. Three basic types exist for electronic positioning systems -- hyperbolic, range/range and range/azimuth.

Hyperbolic systems require three shore-based transmission stations and one shipborn passive receiver. One of the shore based stations, called master, transmits a signal which triggers the other two (slaves). All three signals are received by the vessel's receiver. The principal of hyperbolic position is that a hyperbola is the locus of all points having a constant range difference between two fixed points. In Figure 18, A, B and C are the shore based stations and P is the position of the vessel at the intersection of the hyperbola 3 and 11. Hyperbola 3 is the LOP resulting from the range difference between stations A and C while hyperbola 11 results from the range difference of the vessel between stations A and B. Hyperbolic positioning is divided in two groups according to the principles employed. One method is by measuring the time difference between the reception of the syncronized signals from each pair of stations. In the other method, the phase difference between the received signals is measured. The hyperbolic expansion away from the baseline (line connecting each pair of shore stations) degrades the positional accuracy of these systems.

Range/range systems can be either active or passive and they require only two shore-based stations from which the ranges of the vessel are determined. Active ranging is achieved by measuring either the traveling time of the signal from the vessel's transmitter to the shore station or by measuring phase differences between vessel and station transmitters. Passive ranging is achieved by measuring the time interval between the transmission of the signal from

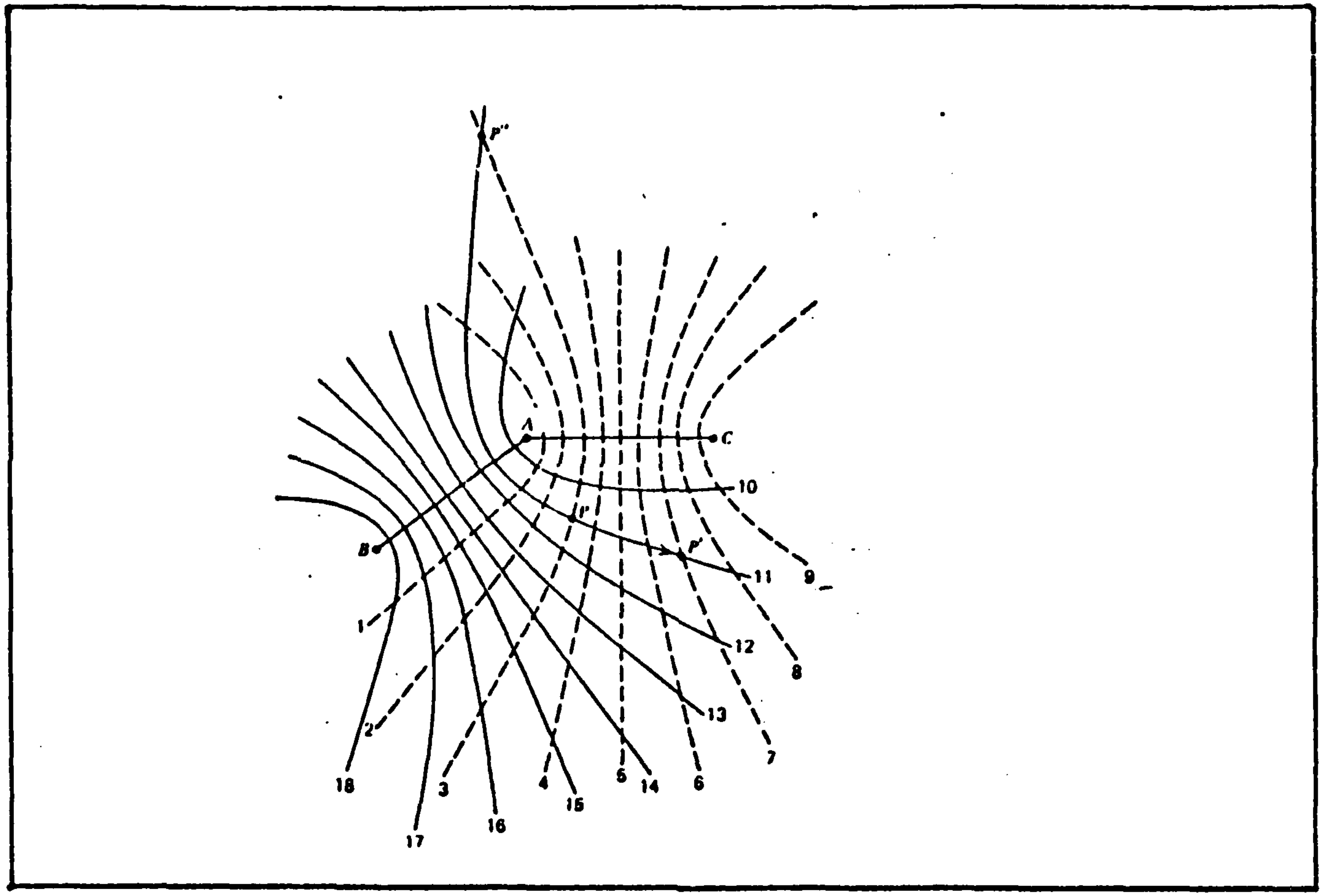

Figure 18. Hyperbolic Positioning System Geometry

the shore station and its reception on the vessel. Figure 19 depicts the geometry of the ranging system.

A third type of EPS is that utilizing the range/azimuth principle. These systems are single user only and require just one shore station to operate. Only two such systems (Artemis and HPR) have been reported in the XVI Congress of Surveyors in 1981 [Ref. 66].

Range/range systems provide a simple circular lattice with no lane width expansion and they require only two shore stations instead of three required for hyperbolic systems. Hyperbolic systems on the other hand have the main advantage that they cover a larger survey area and they have a multi-user capability which is not possible for all ranging systems. The potential positional accuracy of the EPS can be considerably improved by the employment of multiple (more than two) LOPS. Such techniques have not been used extensive for hydrographic survey for charting, but have been successfully used by some offshore engineering firms, especially the oil industry.

The other classification method for electronic positioning systems is according to measurement principle. Again, there are three basic types -- direct wave elapsed time, surface wave phase comparison and UHF systems.

Direct wave elapsed time systems are all called microwave or line-of-sight systems and are generally used over short ranges. Depending on the heights of the

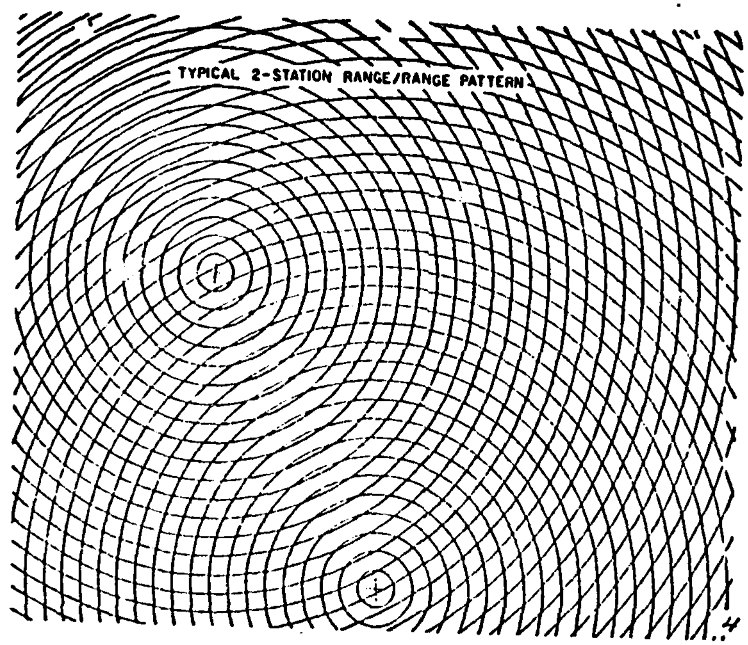

Figure 19. Ranging Positioning System Geometry

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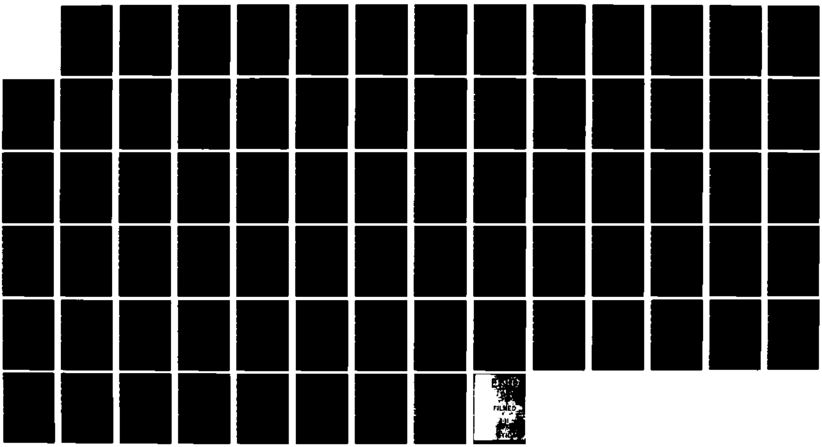

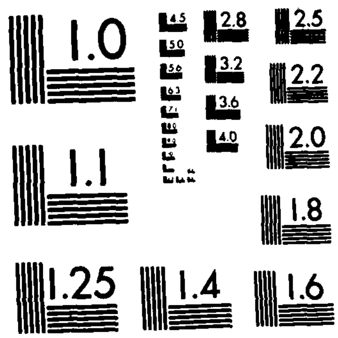

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transmitter and receiver antennas they can operate to a maximum distance of 100 km. Their frequency of operation is generally in the microwave spectrum (3-10 GHz). Some other general characteristics of these systems include their light and easily mobile equipment and their high accuracy over short ranges. Usually the vessel is active and timesharing (multivessel) operation capability is very common. Generally range/range is the fix geometry utilized. Elapsed time is the most used method of measurement, but phase difference, although more expensive, is also used by some systems (Tellurometer, Autotape) in order to achieve better accuracy.

Surface wave phase comparison systems are also called medium range systems and are used for coastal surveys that extend beyond the sight of land. The systems use surface wave electromagnetic propagation. Their accuracy is generally lower than the short range systems. They utilize frequencies of about 2 MHz and they mainly use hyperbolic geometry although ranging mode is also used extensively.

UHF systems are those whose performance falls between the microwave and medium range. They utilize frequencies between 420 and 450 MHz and they propagate through the atmosphere in surface "ducts". Measurement of distance is accomplished by means of coded pulses.

Table XIII shows the EPS's used by the four considered agencies (U.S. NOS, British Hydrographic

TABLE XIII

ELECTRONIC POSITIONING SYSTEMS USED IN DIFFERENT COUNTRIES

	Agency by which it is used				
System Name	U.S. NOS	British Hydrographic Department	CHS	HNHS	Systems Description
Miniranger III	YES		YES		Short range, range/range and (optional) range azimuth, multi-user time sharing
Trispounder	YES	YES	YES	YES	Short range, range/range, multi-user time sharing
Argo	YES				Medium range, range/range (with optional hyperbolic mode) multi-user capability
Hi Fix 6		YES	YES		Medium range, range/range/hyperbolic multiuser
Hydrotrac	YES				Medium range, range/range, hyperbolic
Pulse 8		YES			Range/range or hyperbolic multi-user syste, short range or medium range
Raydist	YES	YES			Medium range, range/range and hyperbolic multiuser
Seafix (Minifix)			YES	YES	Medium range, hyperbolic, range/range multiuser

Department, Canadian Hydrographic Service and Hellenic Navy Hydrographic Service). Table XIV depicts some user results for some electronic positioning systems as they were reported at the XV International Congress of Surveyors, Stockholm, 1977 [Ref. 67]

3. Theodolite Intersections

In this method the survey vessel's position is determined by simultaneous theodolite cuts from two or more stations. The theodolite stations are selected so that good intersection angles (between 30° and 150°) are obtained. The third theodolite, although not absolutely necessary, is usually employed to provide a check, particularly for detached positions. Another reason for the use of more than two theodolites is that they can be used on a continuous basis in a rapidly progressing survey where one instrument and observer at a time will shift position while the other two continue the observations. Synchronization between the observers and the sounding vessel is obtained by radio and is controlled from the vessel. Each observation is recorded at the shore stations. In order to check gross errors, the numbers of fixes are checked at the end of each line and an initial check is obtained before and after each line. Theodolite intersection surveys give very accurate results but they have the disadvantage that they are very slow, require more personnel than other visual methods and plotting is difficult to do during the survey. Sometimes

TABLE XIV

SOME USER'S RESULTS FOR VARIOUS EPS

System	User	Claimed Accuracy	Remarks
Miniranger	Canadian Hydrographic Service	1.5m	RSS range error for distances of 4–9 km.
		9.1m	RSS range error at 15 km.
		9.5m	RSS range error at 21 km.
			All tests static with Tellurometer used for reference positions. Numbers are averages for antenna variations in horizontal plane of 0–80°.
Miniranger	NOAA		Signal reception problems experienced, using antenna mast heights on survey launch of 2m, 4m, 5m, and 6m above water level.
Miniranger	NASA/WFC	2.8 m ± 3.6 m 1.5 m ± 4.4 m 5.0 m ± 5.3 m	Tests of 3 Minirangers, giving mean error and standard deviation about mean with FPS–16 C–Band radars used for reference positions. Survey position for 3rd Miniranger may have had several meter error.
Trisponder	British Transport Docks Board	2.5m, 1.5m	Range errors at 10 km.
		4.3m, 5.0m, 9.0m	Position errors based on sextant determined positions.
Trisponder	NOAA	2.1m	RSS range error for 32 calibrations over distances of 6–9 km, based on sextant fix reference positions.
		2.8m	RSS range error for 28 calibrations over distances of 1–9 km, based on sextant fix reference positions. 4 points at 13 km had average errors of 16m.
Trisponder	NOAA		Tests included measurement of ranging error as function of signal strength (~.27m/db), with resulting variation of 5m in range error between 1 km and 8.5 km.

TABLE XIV (continued)

System	User	Claimed Accuracy	Remarks
Trisponder	Canadian Hydrographic Service	2.6m	RSS range error for distances of 2–9 km.
		9.9m	RSS range error at 15 km.
		14.9m	RSS range error at 21 km.
			All tests static with Tellurometer used for reference positions. Numbers are averages for antenna variations in horizontal plane of 0–80°.
ARGO	AFETR/RCA APL/JHU	19.8m	RSS position error for 170 sample points using Autotape as reference. Initializations found difficult to perform in port due to local multipath problems. System was observed to suffer relative immunity to degradation from atmospheric interference and to have stable signals day and night, even during electrical storms.
Hydrotrac	NOAA/NOS		Lane count repeatability .01–.30 (.8m–23m)
Raydist	NASA/WFC	24 $\pm$ 2.9m 11 $\pm$ 3.5m	Mean error measured with standard deviations about means for "Red" and "Green" baselines. System was not zero set. Reference positions from C-Band radars and accurate to $<$ 3m. Several dropouts and loss of lane count observed. Operation was on edge of lower Chesapeake Bay network.
Raydist–T	AFETR/RCA APL/JHU	27m	RSS fix accuracy, compared to Autotape, for 115 samples. Strong susceptibility to nighttime ionospheric changes and local storms. Pronounced sensitivity to errors as a function of heading which was unexplained.
Raydist	NOAA		Noted drifts in calibration at same point of .2 lanes (9m) over 6 hour period.
Mini–Fix (Seafix)	Canadian Hydrographic Service	–6.0m to 9.8m –3.0m to 5.5m	Variations in calibrations of two chains using Hydrodist for reference lane counts. Corrections made based on monitor recording of pattern variations. Phase lag corrections also made.

[From Hunson 1977]

the sounding lines are run on one of the theodolites and the observer directs the coxswain with signals or by radio.

Theodolite intersections from four shore stations is the visual method usually used by the HNHS. Although the employment of EPS has limited the use of this method, it is still used by the HNHS in about 25% of its surveys. The employment of this method by the U.S. NOS has been limited to fixing the position of floating aids to navigation [Ref. 68].

4. Range-Azimuth

This method is the most popular one for large scale surveys. It involves the combination of a ranging EPS installed on only one shore station with azimuth observations to the vessel via a theodolite from the same shore station.

The survey vessel is usually steered along a circular position line (constant range arc) so that the two lines of position intersect at right angles and give strong fixes. It seems that Trisponder or Mini-Ranger and T-2 theodolite are the most commonly used combinations. For a range-azimuth hybrid system the NOS Hydrographic Manual suggests that

> "... directions or azimuths to the sounding vessel for a position fix shall be read to the nearest 1 minute of arc or better if necessary to produce a positional accuracy of 0.5 mm at the scale of the survey." [Ref. 69].

5. Visual Range and Cut-Off Angle

In this method, which is mainly used in the U.K., the sounding lines are run along preestablished visual ranges or transits which also serve as LOPs (Figure 20). The second LOP is obtained by observing the angle between two shore stations. A rule of thumb for the sensitivity of the range is that the distance between the two marks must be about one-third of the maximum length of the sounding line. The accuracy of the fix, besides the sensitivity of the transit, depends on the cut-off angle. The larger the angle the more accurate the fix is.

6. Distance Line

For very large scale surveys and for relatively short distances offshore, distance line methods are preferable, because sextant angles are insufficient or inconvenient to use. The methods involve the use of a marked line (usually wire) divided into numerous sections, each 2 or 3 meters long. The distance of the sounding launch (or skiff) can then be readily measured when the line is taut. There are three ways in which the distance line can be used. The most common practice is to have one end of the line fixed on the shore while the other end is on a reel on the launch (Figure 21a). Another technique is to suspend the line between two fixed points on shore, across a basin or channel (Figure 21b). In this case the launch proceeds along or below the suspended line and fixes are readily

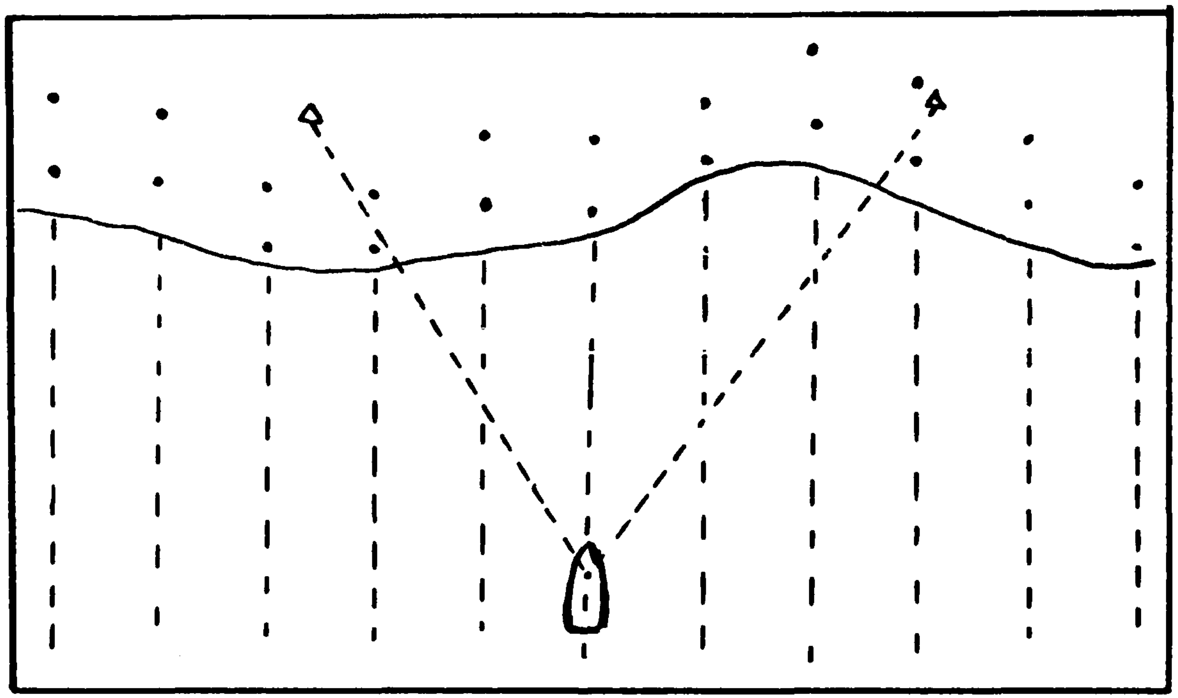


Figure 20. Positioning by Visual Range (Transit) and Cut-Off Angle

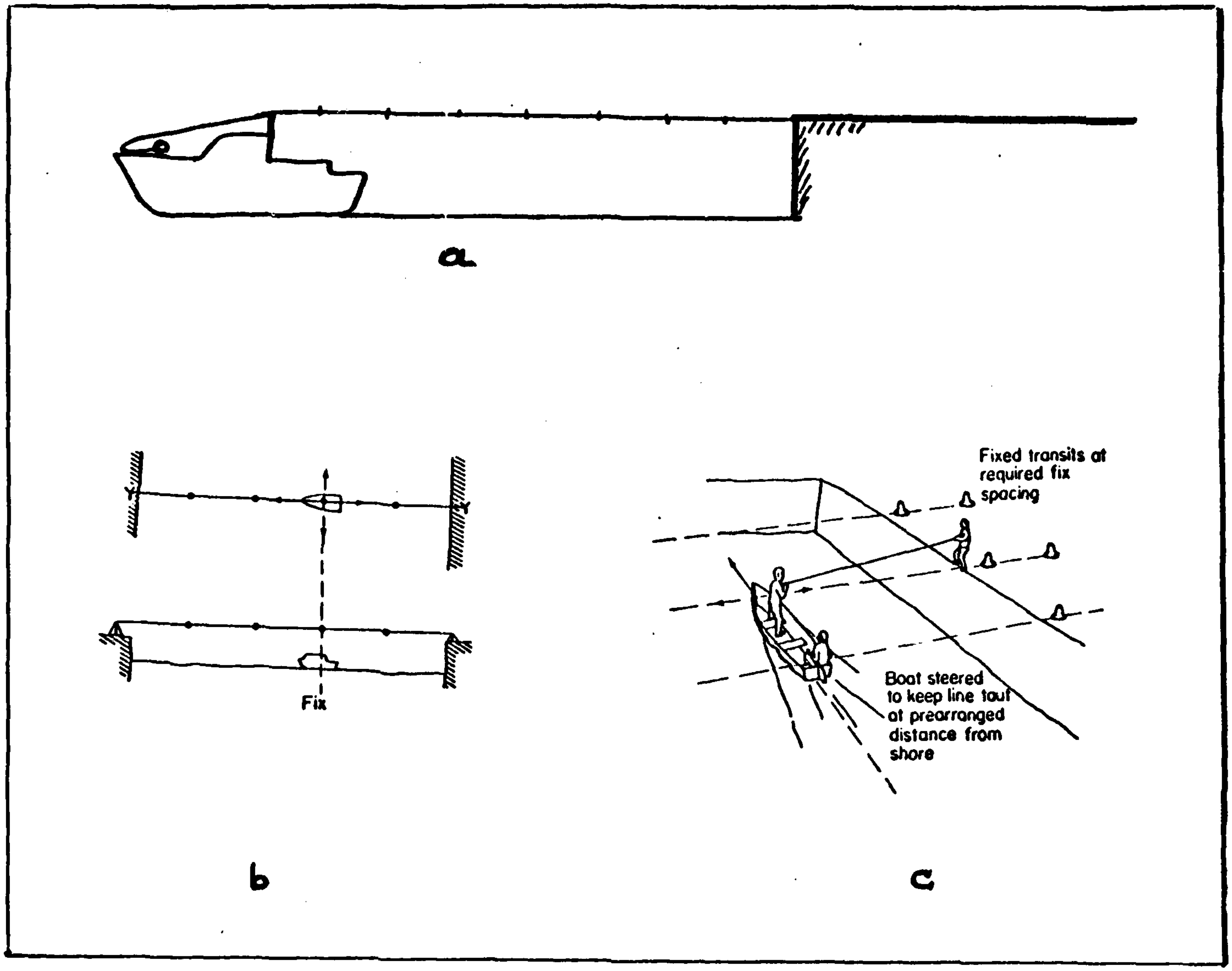


Figure 21. Positioning by Distance Line

determined at the marks of the line. The disadvantage of this technique is that it is inconvenient in busy basins and channels. A third method suitable for close sounding along a vacant quay or dock is to use the line to keep the launch at a fixed distance from the shore (Figure 21c). A second LOP is obtained by a prefixed transit.

Distance line methods are very accurate for distances up to 30 meters when the sag of the wire can be neglected [Ref. 70]. Attention must be paid for the line to be horizontal, otherwise the measured distance will be too great. Although there are ways to calculate the sag correction, the procedure is very difficult to apply to a moving launch. In practice, the sag effect is overcome by simply increasing the length between successive fixes -- instead of fixing every 3 meters, fix every 3.1 meters then every 3.2 meters, and so forth. Using this method of fixing, distance lines can be used for distances up to 150 meters.

The distance line method is used by all four of the agencies considered (U.S. NOS, British Hydrographic Department, Canadian Hydrographic Service and Hellenic Navy Hydrographic Service). The NOS Hydrographic Manual gives an illustrative example for the execution of a "tag line" (distance line) survey and suggests that a sounding line interval of 25 feet with soundings taken at intervals of 25 feet along the line is sufficient for a 1:1200 scale survey [Ref. 71].

7. Subtense Bar

This method is suitable for large scale surveys close to quay walls (up to 160 meters distance). The bar is usually about 7 meters in length and is held vertically with its base at the same level as the observer's eye. The principle is that each observed angle (θ) and the subtended section of the bar yields the distance of the launch from the bar. The method can be used in one of the following two ways:

(1) A fixed angle is used (usually 2 1/2° or 5°) and the bar is marked at intervals representing the ranges subtended by the fixed angle (Figure 22).

(2) The distance off is obtained by measuring the angle subtended by the bar (Figure 23). Usually the angles corresponding to specific distances (20, 40, 60, 80, 100 m) are precomputed and tabulated beforehand.

Usually a visual range or a line of sight determined from the shore by sextant or theodolite are used for controlling the track of the sound boat. The observer on the launch marks the echo sounder record as each predetermined angle subtends the bar or as each distance mark is brought into coincidence with the zero in turn. The position of the launch is the intersection of the range arc and the controlled track line. The survey may be simplified by preplotting the fixes. Then only the soundings on the fathogram need to be marked. A remote echo sounder fixing

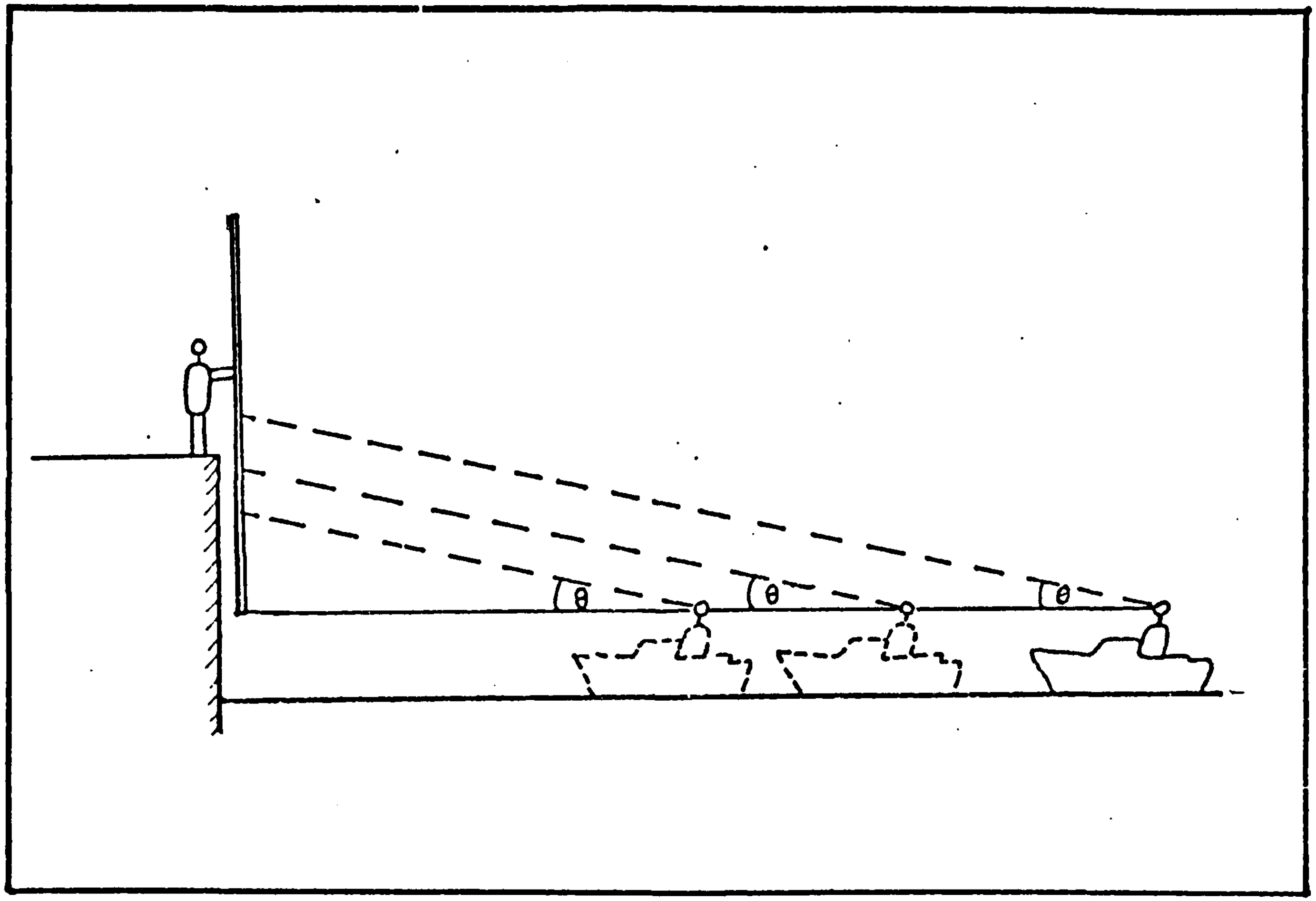


Figure 22. The Subtense Ranging Technique When Observing with a Constant Angle

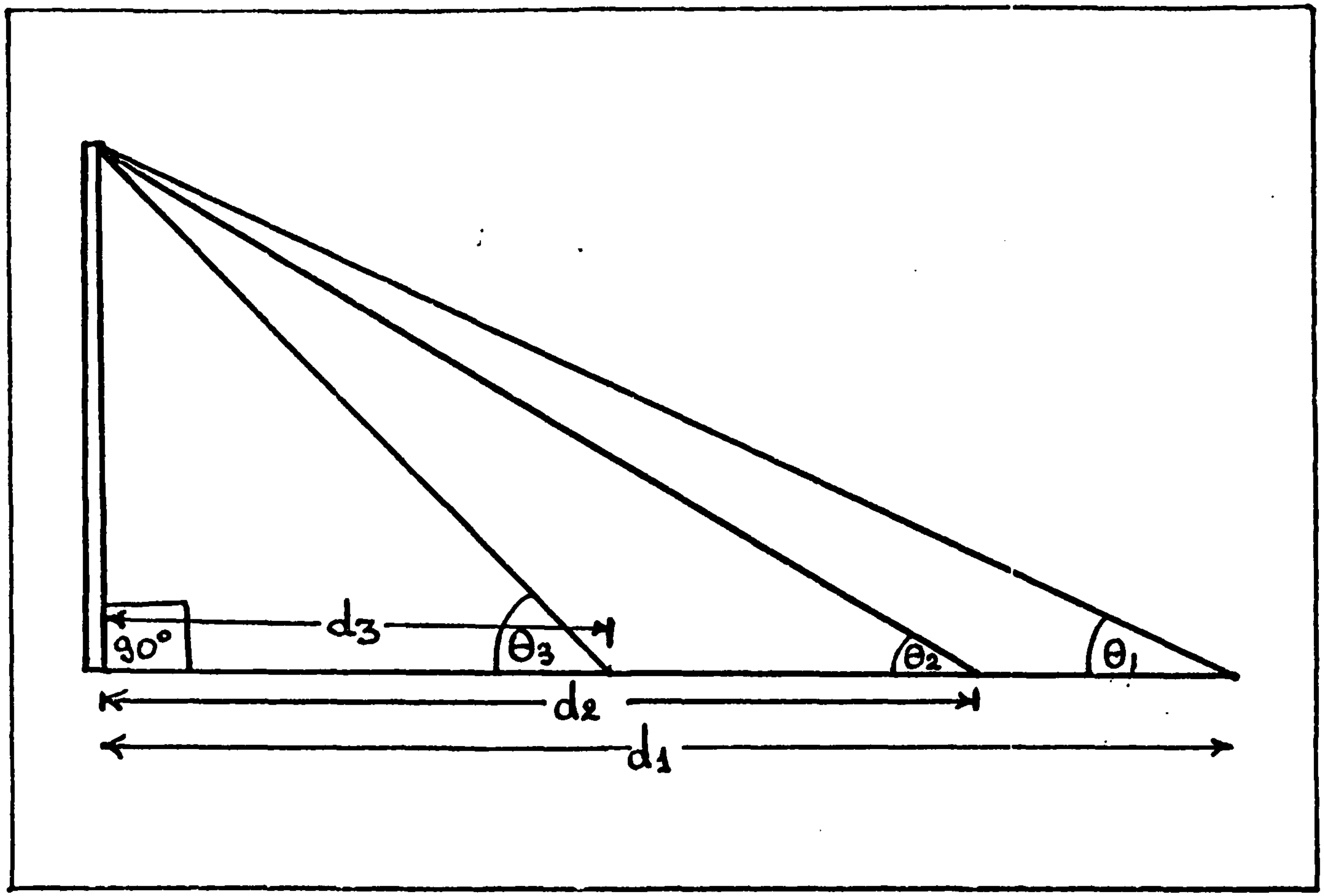


Figure 23. Subtense Ranging by Measuring the Angle Subtended by the Bar

button on the sextant minimizes synchronization errors and eliminates the requirement for an extra man on the echo sounder [Ref. 72]. Potential errors in this method are caused by either a non-vertically held bar or by a difference in height between the observer's eye and the zero of the bar. Another important source of positional error in the subtense bar method is the sextant observational error. Sebbage [Ref. 73] provides the following values of Table XV for the estimation of the resulting positional error corresponding to a 1 minute sextant error. The subtense bar method is used by the British Hydrographic Department, but is used very rarely (if at all) in the U.S. NOS and the Hellenic Navy Hydrographic Service. The Canadian Hydrographic Service utilizes this method with satisfactory results (accuracy ± 2 meters), but only for distances less than 125 meters [Ref. 74].

8. Measured Base and Sextant

This method is also used for large scale surveys close to quays. A measured base is established at right angles to the predetermined sounding lines along the quay (Figure 24). The sounding lines are established by visual ranges that should be perpendicular to the base, equally spaced, and their intersections with the base appropriately marked. The ends of the base are also marked with flags. Sextant angles to the ends of the baseline are measured from the launch to determine it's position on the sounding line.

TABLE XV

SUBTENSE BAR POSITIONAL ERRORS CORRESPONDING TO 1 MINUTE SEXTANT ERROR

Observed Angle	Dist. Off	Possible Error
0° 30'	802 m	53.54 m
1° 30'	267 m	5.94 m
2° 30'	160 m	2.14 m
3° 30'	114 m	1.09 m
4° 30'	89 m	0.66 m
5° 30'	73 m	0.44 m
6° 30'	61 m	0.32 m
7° 30'	53 m	0.24 m
8° 30'	47 m	0.19 m
9° 30'	42 m	0.15 m
10° 30'	38 m	0.12 m

	Observed Angle 2 1/2°		Observed Angle 5°	
Dist. Off	Possible Error	Length of Bar	Possible Error	Length of Bar
160 m	2.14 m	6.99 m		
150 m	2.00 m	6.55 m		
140 m	1.87 m	6.11 m		
130 m	1.74 m	5.68 m		
120 m	1.66 m	5.24 m		
110 m	1.47 m	4.80 m		
100 m	1.34 m	4.37 m		
90 m	1.20 m	3.93 m		
80 m	1.07 m	3.49 m		
70 m	0.93 m	3.06 m	0.43 m	6.74 m
60 m	0.80 m	2.62 m	0.37 m	5.78 m
50 m	0.67 m	2.18 m	0.30 m	4.81 m
40 m	0.53 m	1.75 m	0.24 m	3.85 m
30 m	0.40 m	1.13 m	0.18 m	2.89 m
20 m	0.27 m	0.87 m	0.12 m	1.93 m
10 m	0.13 m	0.44 m	0.06 m	0.96 m

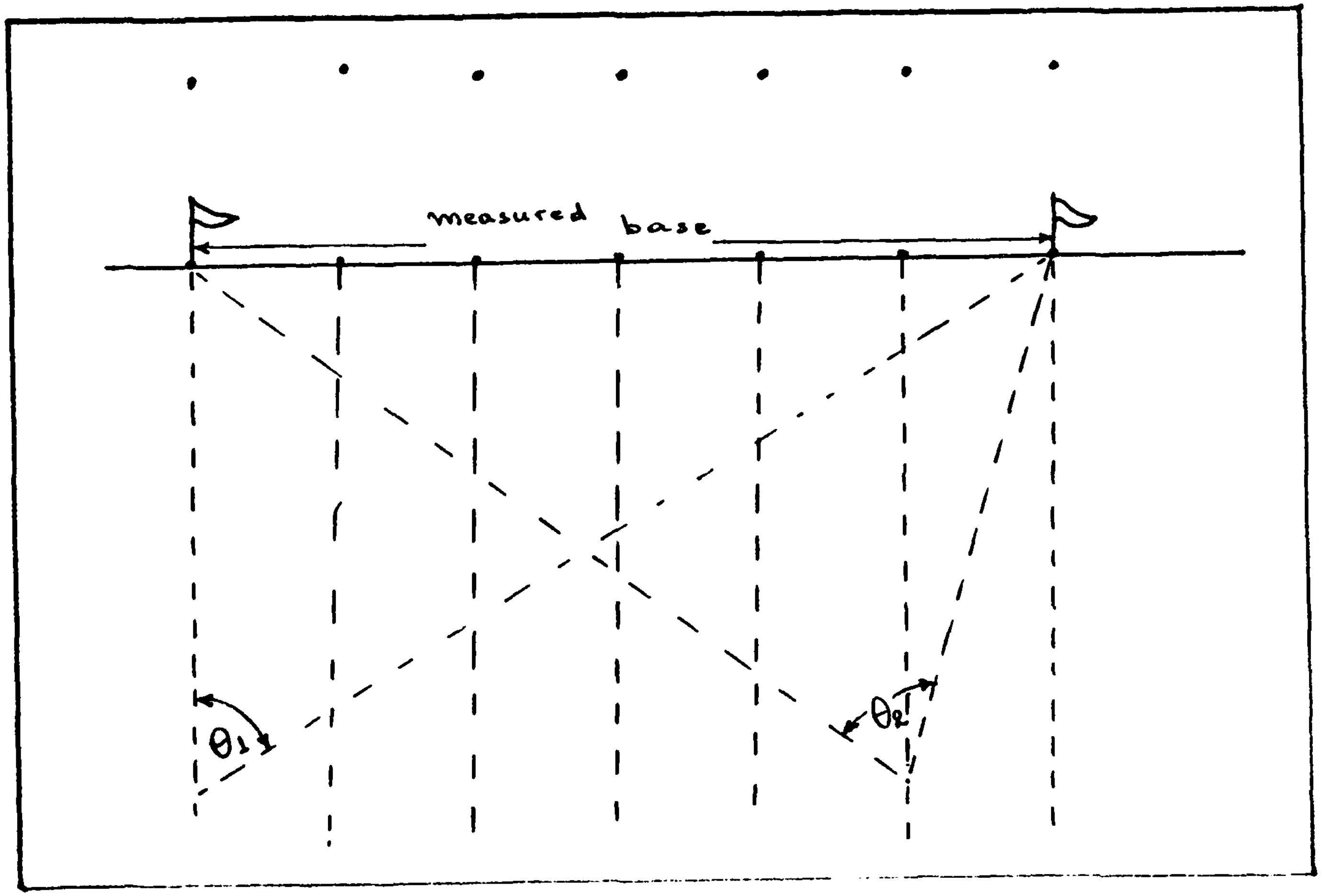


Figure 24. Measured Base and Sextant Positioning Method

The launch is controlled from the shore by theodolite or sextant. Usually the angles corresponding to specific distances are precomputed for each sounding line and the fixes are preplotted. This method is used by the Canadian Hydrographic Service with very satisfactory results [Ref. 75].

9. <u>Transits (Visual Ranges)</u>

This method is suitable for repetition surveys such as channels and dock entrances. The sounding lines are controlled by preestablished transits which also serve as LOPs, while other transits at right angles to the sounding lines give the boat's position at fixed intervals (Figure 25). Although a considerable amount of work is required to set the transits, once they have been established the survey is carried out very easily and only one person is required. The accuracy of this method depends on the sensitivity[17] of the transit, which is shown in Figure 26 and is given by the following formula provided by Sebbage [Ref. 76].

$$S = (D + d/2)\ 2a/d$$

---

[17] Sensitivity of the transit is the distance that the launch is off the transit due to errors in the transit marks.

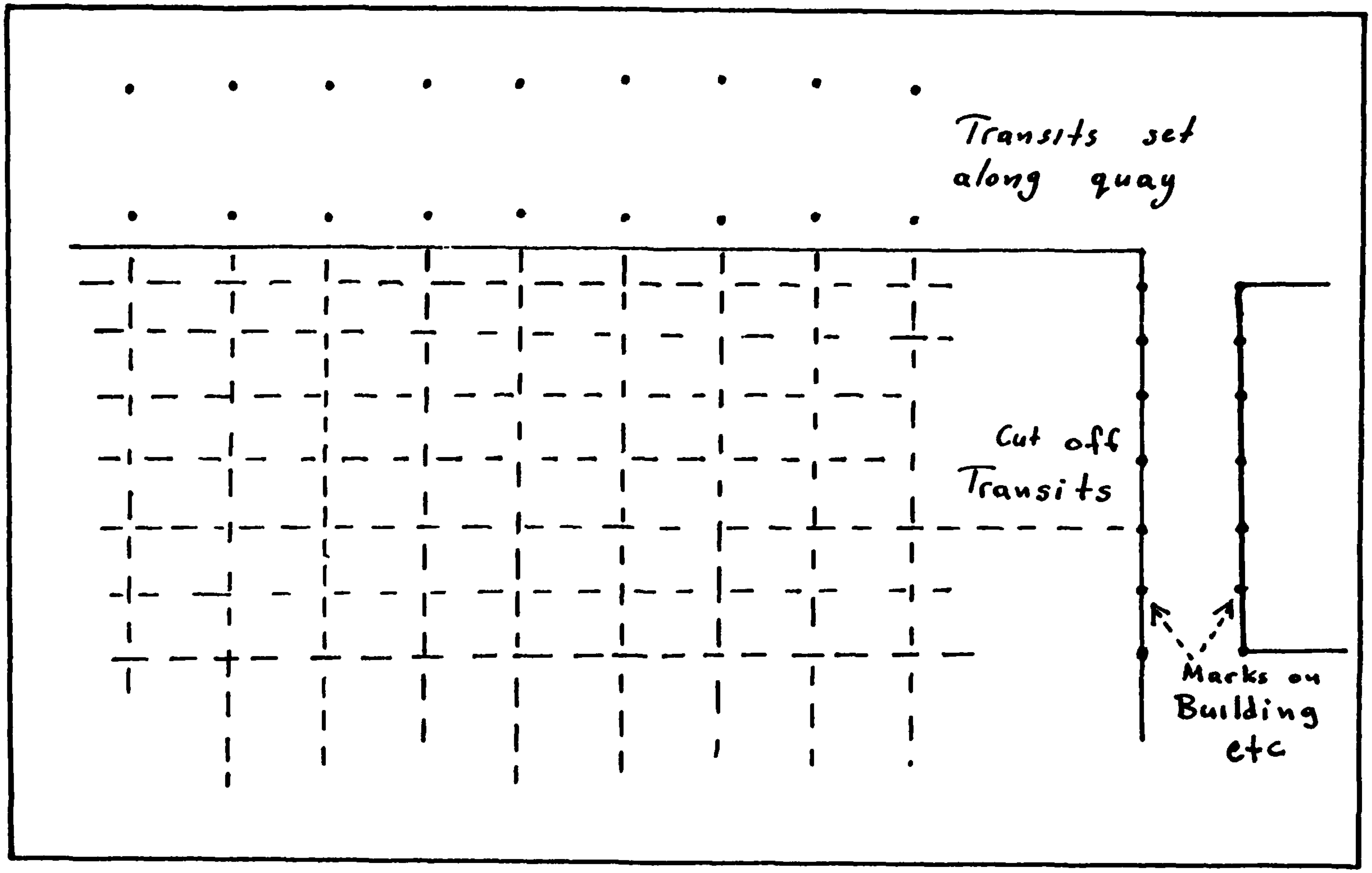
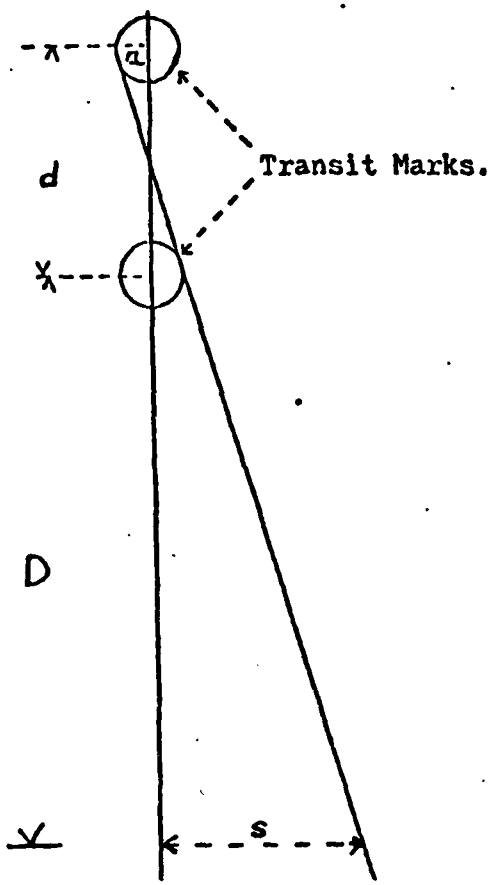


Figure 25. Positioning by Transits



$$S = \left(D + \frac{1}{2}d\right)\frac{2a}{d}$$

Figure 26. Sensitivity (S) of a Transit

where: S = Sensitivity of the transit

D = Distance from the seaward mark

d = Distance between the marks

a = Position error in the transit mark.

10. <u>Other Positioning Methods</u>

The positioning methods presented in the previous sections are not the only ones which the hydrographer can use. Many other methods, mainly combinations of the principles illustrated in this study, are possible and at times more efficient than the described methods. An example is the already mentioned range-azimuth combination. Rockwell [Ref. 77], shows how the CHS used the low cost AGA Gedimeter 120 mounted on a T-2 theodolite to conduct satisfactory large scale hydrographic surveys. In this and in other similar methods, a reflector on the mast of the launch is necessary in order to obtain satisfactory results. Other combinations of positioning methods are possible. Some of these are the combination of a distance range obtained from an EPS with a sextant range visual angle observed from the vessel, or even a hyperbolic LOP with a sextant angle (hypervisual method). Both these methods-combinations are described in the <u>NOS Hydrographic Manual</u> [Ref. 78].

B. DEPTH MEASUREMENTS AND CORRECTIONS

In modern hydrographic surveying, depths are measured almost exclusively by echo sounders. When a lead line or a sounding pole is used, it is usually in very shallow water or over shoals and other submerged features, to verify the echo sounder measurements. Other techniques and methods for depth determination have been tried which promise a new revolutionary change on the present methods of hydrographic surveing because they minimize or even eliminate the operation of the survey vessel. Such techniques are:

(1) Photobathymetry is the technique of obtaining hydrographic data from aerial photographs. This method is already in use by the U.S. NOS but it is still in the development stage. Depths up to 70 feet are the present limits of photobathymetry within NOS [Ref. 79]. NOS has estimated that photobathymetry has a cost benefit of a ratio of 1:6 compared with standard procedures and equipment [Ref. 80]. In the United Kingdom the method is used from helicopters [Ref. 81] and in Canada it is combined with the laser method presented below [Ref. 82].

(2) Laser hydrography. This method is suitable for depths between 2 and 30 meters. This method has already been used in Australia [Ref. 83] and Canada where it is combined with photobathymetric methods to give more accurate results [Ref. 84]. NOS is developing a laser hydrographic system which it hopes to have available in the near future [Ref. 85].

(3) The use of satellite imagery like LANDSAT data is another promising method. This approach has already been used by the U.S. Defense Mapping Agency to add and correct bathymetric data on some old charts [Ref. 86]. Depths up to 40 meters were measured with typical accuracies of 10% in 22 meters depth. The main utility of this method is to easily locate shoals and reefs.

Although the above sophisticated methods for obtaining bathymetric data have been used, the echo sounder on a vessel or launch continues to be the main tool of the hydrographic surveyor. Echo sounders determine depth by measuring the two way travel time for an acoustic pulse to travel from the transducer to the bottom and back to the transducer again. The measured time is converted to distance assuming a known fixed sound velocity in the seawater. Depths observed by echo sounder include several potential errors for which they must be corrected. Usually the required corrections are:

1. Heave Correction for Wave Action

This correction compensates for large vertical displacements of the survey vessel during rough sea conditions. It is difficult to apply except when soundings are scaled from an echogram over a regular bottom. In digital echo-sounders the problem is more complicated unless an analog recording of the depth is also available. A promising solution to the problem is the improvement of the computer assisted (automated) survey methods. Already there are two different systems available providing very satisfactory results for short period waves [Ref. 87]. One system computes the vertical displacement of the vessel by integration of the output of accelerometers. In the other system developed by Navitronic in Denmark, the vertical displacement is computed from the doppler shift of the sonar signals reflected from the bottom.

2. Echo Sounding Instrument Correction

This error is dependent on the specific type of equipment used. Instrument errors are found almost exclusively in analog echo sounders. Initial and phase errors are examples of instrument errors [Ref. 88]. The initial error is found in echo sounders using lined recording paper and is caused by the noncoincidence of the leading edge of the echogram with the zero line of the recording paper. The phase error is a disagreement of soundings common to more than one scale of the analog recorder.

3. Transducer's Separation and Draft

The transducer's separation error is due to the horizontal distance s_1 between the transmitter and receiver of the transducer. The error is the difference between apparent depth (r) and true depth (d) and is equal to:

$$\text{separation error} = r - \sqrt{r^2 - 1/4\ s^2}$$

These errors are illustrated in Figure 27. The transducer's draft (h) is referred to the water line when the vessel is stationary. It is measured via permanently marked points on the hull near the deck. Since most modern transducers do not have a separation between transmitter and receiver, the separation error usually does not exist.

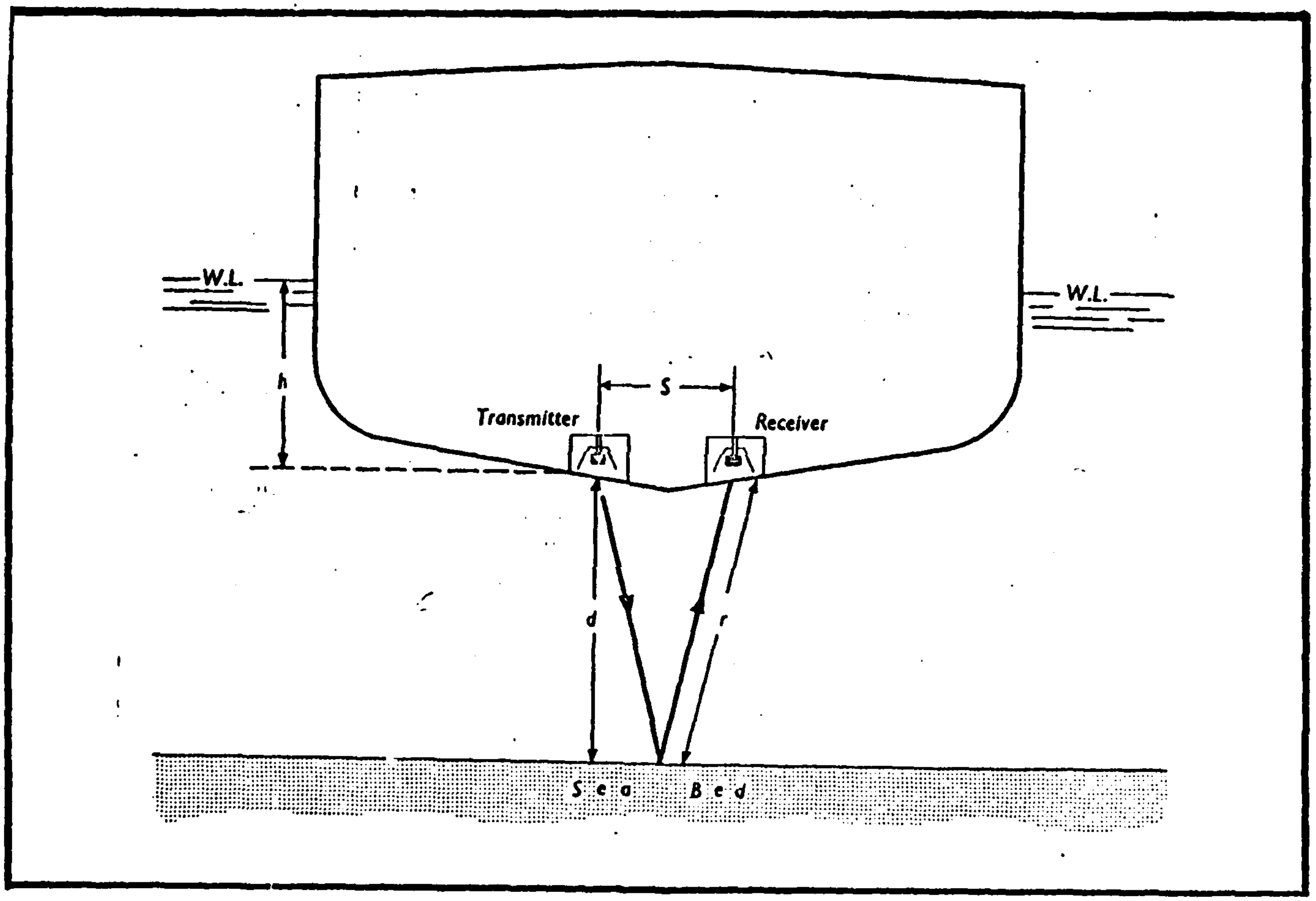


Figure 27. Transducer Separation
[From the AMHS]

4. Settlement and Squat

When the survey vessel is underway, particularly in depths less than seven times its draft, the effects of settlement and squat must be measured and appropriate corrections must be applied to observed soundings. Settlement is the difference in elevation of a vessel when underway versus when stationary, but is not a change in the vessel's draft. Squat is due to the change in trim of the vessel when underway compared to when it is stationary. Settlement is greater at shallower depths (less than 10 times the vessel's draft) and higher vessel speeds. Squat depends on the vessel's speed, but its effect is minimized if the transducer is mounted at the vessel's vertical pivot point. Since it is very difficult to separate the effects of settlement and squat for a vessel underway, the combined effect of both is determined and applied as one depth correction. The measurements should be made over flat even bottom near either high or low tide, when tide heights change slowly. In either case tidal changes must be taken into account.

Probably the most accurate method to measure settlement and squat is that recommended by NOS Hydrographic Manual [Ref. 89] with a leveling instrument, setup on shore. Observations are made on a levelling rod aboard the vessel when stationary and underway at a predetermined point from shore. The difference in the observations gives the settlement and squat of the vessel at that speed. This is repeated at various speeds to obtain a complete table of settlement and squat corrections. Another method also recommended by the NOS Hydrographic Manual involves the comparison of two soundings of the vessel over the same point, one with the vessel stationary, the other with the vessel underway. A moored buoy is necessary to ensure that the vessel measures the depth at the same point each time. The combined effect of settlement and squat may in some cases reach 1 foot [Ref. 90].

The AMHS suggests the following method for the measurement of settlement and squat in boats. The method requires a flat smooth bottom and calm sea conditions. The two boats compare depths when both are at rest side-by-side, which should agree exactly. One boat remains stationary and the other passes close by. Each boat observes the depth with the resulting difference being the settlement and squat at that speed. Although less accurate than the NOS leveling method, this method has the advantage of not requiring any tidal correction.

5. Sound Velocity Corrections

Echo sounder depths are subject to errors due to the difference between the calibrated echo sounder sound velocity and the actual value in the survey area. Many methods can be used in order to determine these corrections, the most important ones being the bar-check method, direct sound velocity measurements via velocimeters, and finally, indirect determination of the sound velocity by measuring temperature, pressure and salinity.

Occasionally some less accurate methods are used, usually in deep waters. These methods involve the computation of the sound velocity from historical data for different regions, seasons and depths. Echo Sounding Correction Tables, which replaced the old Mathews tables [Ref. 91], are sometimes used by the British Hydrographic Department and Canadian Hydrographic Service, is one method which provides velocity correctors to a standard echo sounder velocity of 1500 m/s.

a. The Bar Check Method

The bar check method is a simple method for obtaining depth corrections for the combined effect of sound velocity variations, instrument errors and the transducer static draft. The method consists of lowering a bar at various known depths below the echo sounder transducer and simultaneously observing the echo sounder depth. The bar is lowered to the known depths via two marked lowering lines. Under ideal conditions (calm sea, no wind or current) it may be possible to obtain satisfactory results to 15 fathoms [Ref. 92;93]. The NOS Hydrographic Manual and the AMHS give detailed descriptions of the procedure including the required equipment.

Of additional interest is a variation of this method developed and used by the Canadian Hydrographic Service [Ref. 94] illustrated in Figure 28. This variation uses an inverted weighted cone attached to a single wire and lowered by only one person using a hand winch. The deflection of the bar check apparatus from the vertical is minimized because the cone and the wire are very heavy and their cross section area is very small. Hence, this method can be used in quite deep waters. Also, additional targets (flat, round aluminum plates) can be set at prescribed depths so that a complete bar check can be performed in one only echo sounding transmission. The lowering of the cone from a ship allows the rapid bar checking of several launches. Each passes over the lowered cone and targets, and can obtain a complete bar check in only one pass.

b. Oceanographic Methods

The speed of sound through seawater can also be determined indirectly by measuring the temperature, pressure and salinity of the seawater. Many indirect methods exist to determine the sound velocity, the most popular being Wilson's equation:

$$C = 1449.14 + V_t + V_p + V_s + V_{stp}$$

where: $V_t = 4.5721t - 4.4532 \times 10^{-2}t^2 - 2.6045 \times 10^{-4}t + 7.9851 \times 10^{-6}t^4$

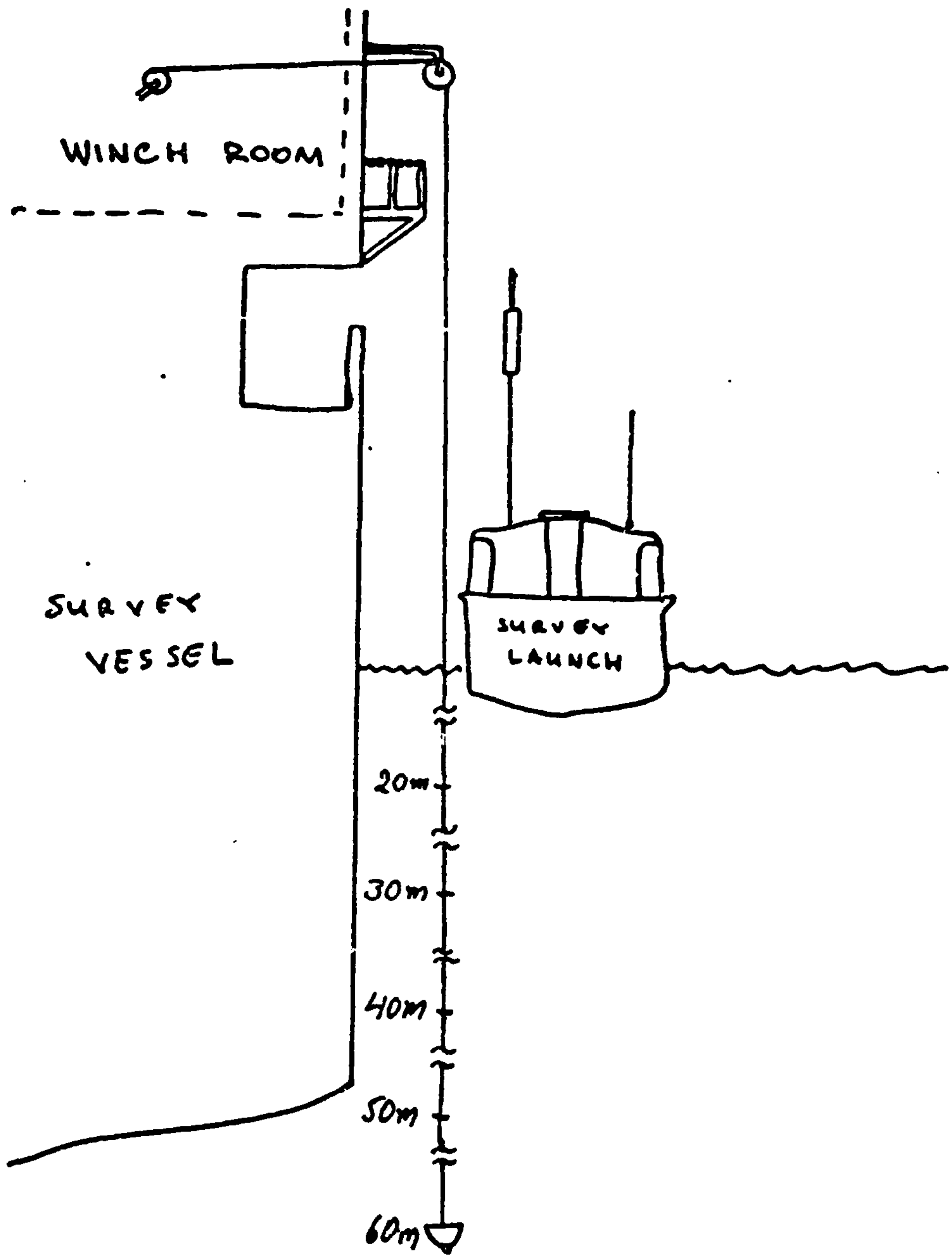


Figure 28. The Canadian Variation of the Bar Check Method

$$V_p = 1.60272 \times 10^{-1}p + 1.0268 \times 10^{-5}p^2 + 3.5216 \times 10^{-9}p^3 - 3.3603 \times 10^{-12}p^4$$

$$V_s = 1.39799\,(S - 35) + 1.69202 \times 10^{-3}\,(S - 35)^2$$

$$\begin{aligned} V_{s+p} = {} & (S - 35)\,(-1.1244 \times 10^{-2}t + 7.7711 \times 10^{-7}t^2 + 7.7016 \times 10^{-5}p - 1.2943 \times 10^{-7}p^2 + 3.1580 \times 10^{-8}pt \\ & + 1.5790 \times 10^{-8}pt^2) \\ & + p\,(-1.8607 \times 10^{-4}t + 7.4812 \times 10^{-6}t^2 + 4.5283 \times 10^{-8}t^3) \\ & + p^2\,(-2.5294 \times 10^{-7}t + 1.8563 \times 10^{-9}t^2) \\ & + p^3\,(-1.9646 \times 10^{-10}t) \end{aligned}$$

t in ^oC, p in kg/cm^2, S in (o/oo), C in m/s

According to Urick [Ref. 95]:

> "The Wilson formula has received general acceptance as the most accurate empirical expression for sound velocity as a function of temperature, depth and salinity."

The determination of sound velccity by the above method (or by a velocimeter) refers to a specific depth. In echo sounding, the average velocity over the complete sounded depth must be determined. The NOS method of layer corrections [Ref. 96] is an efficient and easy way to estimate the average sound velocity over the whole water column. According to this method the water column is divided into a number of layers of varying thickness and the sound velocity is calculated for each layer mid-depth. If the oceanographic measurements do not correspond to the preselected mid-depths the required values are scaled from the plotted velocity profile. Knowing the value of the sound velocity at each layer mid-depth, a correction factor for each layer is calculated by the formula

correction factor =

where: A is the actual velocity at the layer mid-depth.

C is the calibrated velocity for the echo sounder.

The calculated factors are multiplied by the layer thickness to yield the layer corrections. The layer corrections are then summed algebraically to give the correction applicable over the whole water column to the bottom of each layer. The resulting corrections are usually plotted as a correction versus depth curve for convenient use.

The selection of the layer thickness is, generally, based on the existing temperature gradient and need not be the same throughout the whole water column. NOS experience has shown that 10 meter layers for the upper 200 meters, 40 meter layers from 200 to 400 meters and 400 meters for deeper depths usually give satisfactory results.

Sound velocity corrections obtained by the above method can be combined with bar check results to yield even more accurate corrections. The method is described in the NOS Hydrographic Manual and involves the plotting of both correction curves (bar-check and oceanographic) on the same plotting paper and (Figure 29). The two graphs should be identical but displaced a distance d which represents the combined residual error plus the transducer's static draft which is applied separately as another sounding correction.

6. Tide Reductions

The observations for tide reductions to be applied to the measured depths are stated in the IHO recommended standards. In practice, the procedure of both the U.S. NOS and the British Hydrographic Office are to first apply in the field an approximate tide correction derived from either a few hours tide observations or from predicted tides for the area. Corrections for actual or real tides are applied later when all of the required tidal observations have been completed.

The U.S. NOS and the Canadian Hydrographic Service are experimenting with a tide telemetry system [Ref. 97] which will provide real time tide information to their automated systems. Such data is transmitted to the vessel

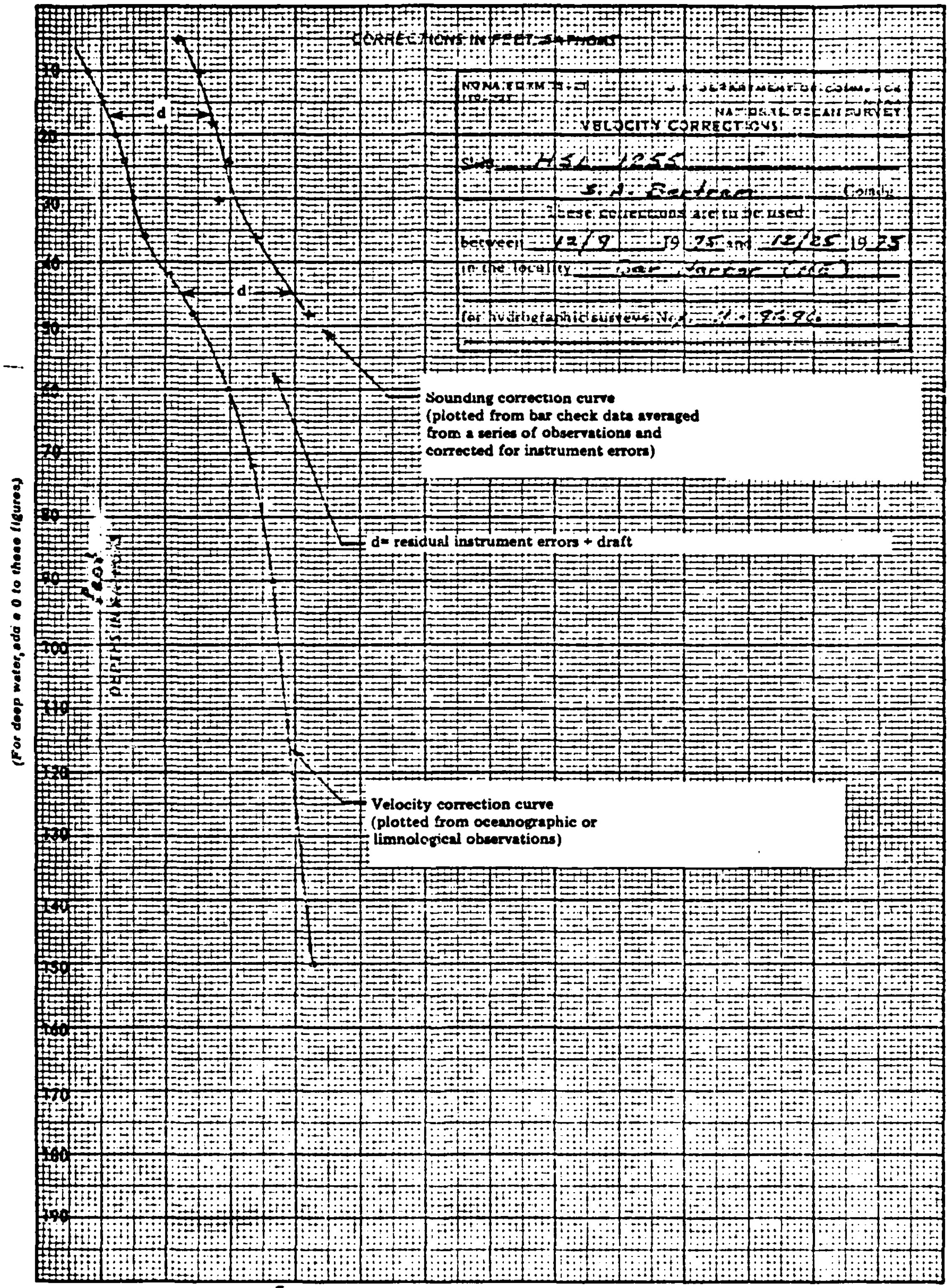


Figure 29. Velocity Correction Curve with Combined Observations [From the NOS Hydrographic Manual]

from special tide gages on shore in close proximity to the survey area.

C. SOUNDING AND SEARCH TECHNIQUES

The hydrographer strives to achieve his goal of adequately delineating the bottom topography using the resources available in the shortest period of time. To accomplish this he must plan and run an efficient pattern of sounding lines which depend on line spacing interval and other factors. As a general rule, suggested by both the AMHS and the NCS Hydrographic Manual, sounding lines should be straight, equally spaced and normal to the depth contours. In the case of an electronic positioning system (without automation), sounding lines may be planned and easily run on circular or hyperbolic arcs. The importance of running straight (or regular curved) sounding lines is that they provide a check for the adequate coverage of the surveyed area while the sounding process takes place. Another reason for the use of regular sounding lines is that they give an estimation of the track which the survey vessel followed between successive fixes.

Despite the fact that hyperbolic arcs are easily followed by the survey vessel, parallel straight line surveying accomplishes the same coverage of surveyed area with fewer and less complicated lines. Straight line hydrography increases the productivity of the survey to about 25 to 30% over that run while following hyperbolic arcs [Refs. 98;99].

In some surveys, especially large scale ones, sounding lines serve as position lines also. In such cases the survey vessel (or launch) is precisely kept on the planned sounding line, either by means of a preestablished transit (visual range) or following the directions of an observer who is sighting on the vessel from the shore with either a sextant or theodolite. Instead of parallel sounding lines, short radiating lines are most efficient in small bays, at the edges of piers and wharfs, around small off-lying islets, at capes or where a significant topographic feature on the shoreline occurs.

Interlines are run between two already run sounding lines. If after the first fix of a new sounding line it is realized that the spacing is greater than the maximum permissible, no attempt should be made to close the spacing because a non-parallel, non-straight, unacceptable sounding line will result. Instead the line should be run parallel to the previous one and an interline should be inserted thereafter. Another case where interlines are run is when a shoal is suspected. In this case enough interlines are run until the suspected shoal has been totally identified or disproved.

As was reported at the XV International Congress of Surveyors [Ref. 100], it is possible to record more than one sounding line per survey vessel by employing more than one transducer. Such methods have been successfully used in many countries, such as Denmark, where five sounding lines are obtained from 5 towed transducers. In the Netherlands, two external transducers are used. In Sweden another technique is used which involves a number of satellite launches (up to eight). These maintain their position relative to the main surveying vessel and transmit the collected depth data to technicians aboard the vessel.

The conventional sounding line spacings discussed in the previous section on hydrographic specifications, can be expanded to increase the productivity of a hydrographic survey if multibeam or dual frequency echo sounders are used. Multibeam echo sounders use a set of multiple narrow beam transducers to obtain the coverage of a very wide beam while maintaining the resolution of each individual narrow beam. Such systems are the Bo'Sun System used by the Canadian Hydrographic Service and a slightly modified version called the Bathymetric Swath Survey System (BSSS) which is used by the U.S. NOS. Both systems utilize 21 narrow beams (5° each) so that the effective total beam width is 105° and the swath coverage is 2.6 times the depth. In 30 meters water depth these systems can accomplish 100% coverage with a sounding line spacing of 78 meters. For deeper water, to 11,000 feet slant range, the Sea Beam Swath System may be used. It utilizes 16 narrow beams (2 2/3° each) to create an effective beam width of 40° and yield a coverage area of 0.75 times the depth. The use of dual frequency echo sounders is another way to expand coverage for one sounding line. The dual frequency echo sounder utilizes two sufficiently different frequencies for concurrent sounding with two beams, one narrow and one wide. Dual frequency echo sounders can be satisfactory employed to increase regular line spacing and ensure peak detection between them. NOS has recently purchased dual frequency echo sounders to be used as standard equipment on all hydrographic survey vessels.

The employment of side scan sonar during hydrographic surveys is a valuable tool for the detection of wrecks and obstructions [Ref. 101]. Such techniques are used systematically by the British Hydrographic Department and the Canadian Hydrographic Service and to a lesser extent by the U.S. NOS and the HNHS. The British Hydrographic Department usually employs side scan searches in two ways, either with a 100% or with 20% overlap with adjacent sweeps strips. Another way to search for the detection of submerged obstructions used by the British Hydrographic Department and occasionally by the HNHS is by directional sonar search usually used for the detection of submarines.

When using conventional echo sounding techniques for the detection of suspected shoals, reported wrecks and other submerged dangers to navigation, several possible searching patterns exist. The AMHS suggests three basic search patterns depicted in Figure 30. The star search, which is also employed by the CHS [Ref. 102] and the HNHS, requires a buoy on the suspected shoal. The star search has the advantage of crossing the depth contours at right angles but it is very difficult to change while it is conducted. The spiral box search covers the area quickly and evenly and it is especially recommended when sonar sweeping is used. Spiral searches are also used by the HNHS. The rectangular search pattern is the most commonly used one because it not only covers the ground quickly and evenly, but also it can be easily changed while the search takes place.

The employment of side scan sonar, dual frequency echo sounders, and multibeam echo sounders has greatly reduced the use of the traditional sweeps. However, they are still used for the final and most accurate detection or disproval of shoals and obstructions. The most accurate sweeping technique is the conventional drift sweep recommended by the GIHS [Ref. 103]. The wire sweep [Ref. 104], modified trawl sweep and pipe drag [Ref. 105] have been adopted by the U.S. NOS. These have the advantage that they can be performed more quickly and do not depend on tidal streams or currents.

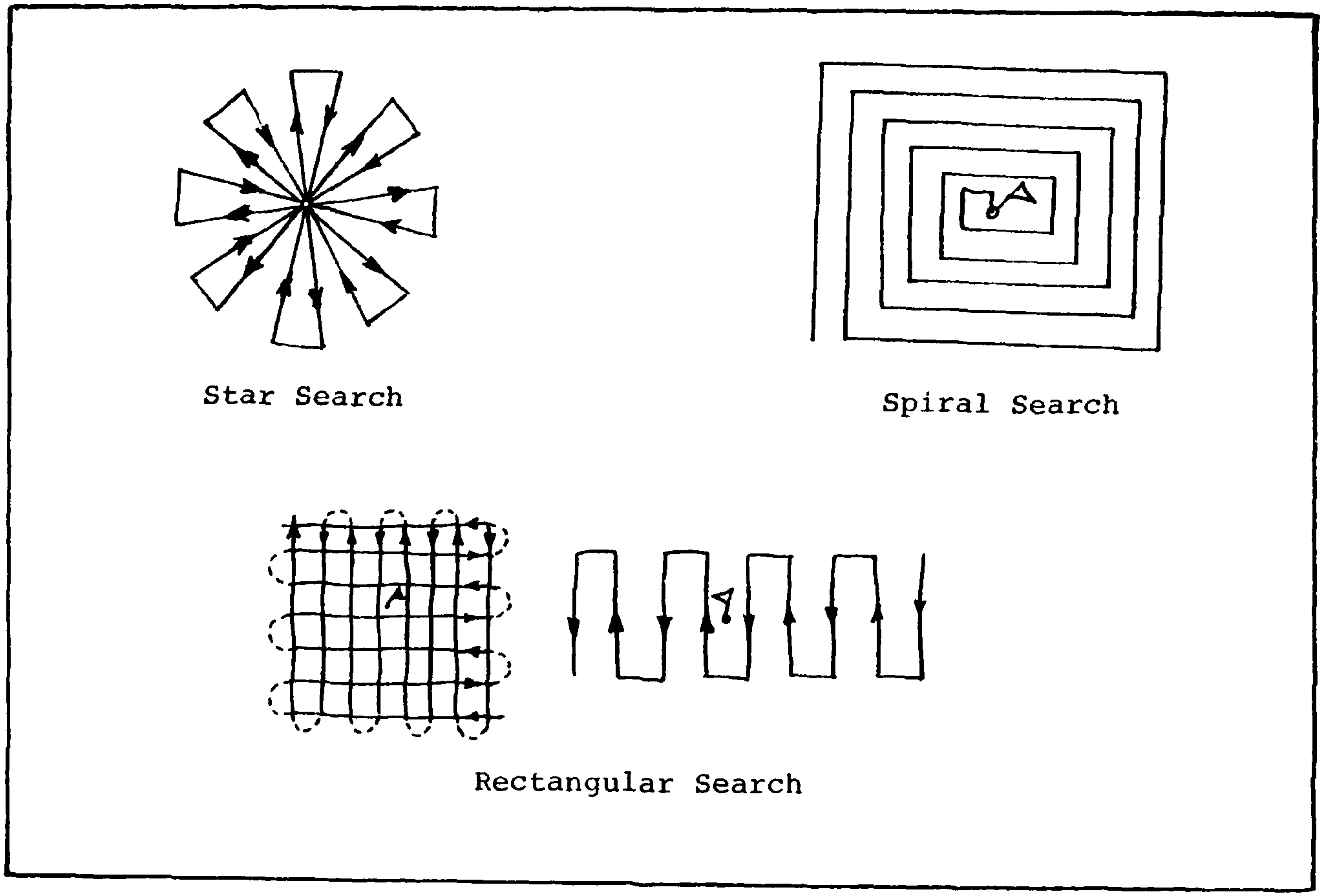


Figure 30. Various Searching Patterns
[From the AMHS]

D. COMPUTER ASSISTED (AUTOMATED) METHODS

The widespread evolution of computers during the last two decades has resulted in a revolutionary change in the methods of hydrographic surveying. The impact of the implementation of computers (automation) in hydrographic surveying can be compared with that of the conversion from lead lines to echo sounding, or to the introduction of electronic positioning systems. The main advantages of automation in hydrography are cost effectiveness, time effectiveness and reliability effectiveness. A detailed analysis of the above benefits of automation was presented at the XV International Congress of Surveyors [Ref. 106].

The basic functions of a typical automated system are the determination of the vessel's position while sounding, the measurement of depth at each determined position and at intervals along the sounding line to the next position, and the recording and/or graphical representation of the above information. Although the capability exists for automated systems to improve the acquired position accuracy by the use of multiple lines of position (more than two), such techniques are not usually employed for regular hydrographic surveys conducted for the benefit of navigation purposes. Source systems provide steering information for the helmsman to maintain straight lines while surveying. Automatic compensation for the heave effect has been successfully applied in a number of cases, but only for short period waves. Two kinds of such systems are available [Ref. 107]. One system employed in the U.S. and the Netherlands computes the vertical displacement of the vessel by integration of the output of accelerometers. In the other system developed by Navitronic in Denmark, the vertical displacement is computed from the doppler shift of the sonar signals reflected from the bottom.

One of the major problems in automated hydrography is the accurate measurement of depths in digital form. The selection and scaling of soundings from analog echograms is easily done by humans, but somewhat difficult with automated techniques. False echoes, noise and interference cannot be easily differentiated from the sea bottom by electronic instruments whereas it is a simple task for humans. For these reasons, some agencies like the German and Swedish Hydrographic Services derive digital depths in a semi-automatic manner by digitizing the echogram with a manually operated pen follower or graticule [Ref. 108].

Some problems appeared with the installation of automated systems in small launches. The power to run the system created the need for an additional electric generator on the launches of the NOS [Ref. 109] and the CHS [Ref. 110]. Although a specially designed launch can reduce the problems and increase the effectiveness of the installed automated system (as was shown in the case of the U.S. NOS "Jensen Boat" [Ref. 111]), the employment of microprocessors[18] seems to be the best solution to the problems of power requirements, size, weight and cost, encountered with the use of minicomputers. Microprocessors have been successfully employed for systems on launches by the Canadian Hydrographic Service [Ref. 112].

There seems to be a difference of opinion whether automation should be restricted to data acquisition only during the sounding process or if some on-line (real time) processing should be included. The present hydroplot system used by the U.S. NOS does the majority of processing on-line [Ref. 113] while other agencies concentrate their processing activities off-line. A new automated system is being developed by the NOS which will possibly eliminate some of the on line data processing. Plans are to employ a digital acquisition system (DAS) on launches with a central data processing system (DPS) on the mother ship. Table XVI shows the capabilities of the various automated systems of the considered agencies.

---

[18] Microprocessor: A microcomputer central processing unit (cpu) integrated on a chip.

TABLE XVI

AUTOMATED SYSTEMS USED BY DIFFERENT AGENCIES

Agency	System Used	Underway Processing Features						
		Depth Digitization	Navigation	Sound Selection	Coordinate Conversion	Heave Compensation	Vessel Position Plot	Initial Sounding Plot
U.S. NOS	NOS Hydroplot	X	X	X	X		X	X
British Hydrographic Department	Marconi Hydroplot	X	X	X	X		X	X
Canadian Hydrographic Service	HYNAV							
	PMAAS	X						
	INDAPS	X	X		X			
	NAVBOX (HYNAV)	X	X		X			
	Intergrated off-shore system	X	X	X	X		X	X
Hellenic Navy Hydrographic Service	AUTOCARTA	X	X	X	X	X	X	X

VI. CONCLUSIONS AND RECOMMENDATIONS

From the examination of the presented methods for the establishment of horizontal control for hydrography, it is evident that triangulation and traverse are the main methods used. There seems to be no agreement among the various agencies as to which of these two methods is mostly used. The U.S. NOS does the majority of its horizontal control surveys for hydrography with traverse (about 90%) [Ref. 114] while the HNHS concentrates on triangulation. The main factor for the selection of one or the other method depends on the geographical configuration of the surveyed area and the availability of good EDM's. When many islands exist, triangulation is probably the best solution, but when no islands exist and the coastline tends to be even, traverse is the most appropriate method. Trilateration itself is not used by any country for the establishment of horizontal control for hydrography, but baselines are occasionally measured to strengthen a weak triangulation configuration and provide additional checks on the angular measurements. Every agency agrees that its specifications do not ensure that the required accuracy standards will be met:

> Canadian Specifications: "At best they are a general guide only and must be used with caution." [Ref. 115]

> NOS Specifications: "... an absolute guarantee cannot be given that a particular standard will be met if all stated specifications are followed ...". [Ref. 116]

> British Specifications: "Common sense and judgement must be used in deciding exactly what to do in a particular case." [Ref. 117]

The only way to make sure that the required accuracy standards for horizontal control are met, is to perform a rigorous analysis of the results of a survey, usually via the least squares method using a large computer. This procedure has the disadvantage that it must be done after the field work. In situations where data are inconsistent, at least some field measurements may need to be repeated. Horizontal control in different orders is based on the relative accuracy between any two stations. The relative accuracy between two stations is usually expressed as a ratio of their distance. This is the way horizontal control is classified in the United States and many other countries. In Canada, horizontal control is classified into different orders of accuracy in a peculiar way through the concept of confidence region.

The Canadian specifications for horizontal control are of particular interest for the following reasons:

(1) They are based on practical experience as well as on the results of analysis of networks.

(2) The adopted concept of confidence region, permits the prediction of the accuracy of a prospective survey. The design of the survey can be changed to increase the probability of success.

The weak point of the Canadian specifications is that they focus on idealized networks only, like those in Figures 6 and 7, which are very unlikely to happen in reality. This disadvantage can be eliminated by applying the rules of thumb suggested by the British Hydrographic Department.

Emphasis must be given to the suggested ways to estimate whether the configuration of a horizontal control survey network is a strong figure or not. The NOS method using the concept of strength of figure is not very valuable now since little "pure" triangulation is now done [Ref. 118]. Modern electronic distance measuring equipment, although very expensive, provides redundant observations by measuring additional lines to strengthen the figure of a triangulation net. The tendency for modern horizontal surveys is to become a mixture of triangulation, trilateration and/or traverse in the sense that the principles of one technique are used to strengthen another. The concept of "strength of figure" is not applicable in these cases. Other more complex techniques are adopted to check the strength of the net, such as side equations explained in Appendix A.

The British specifications for the observation of horizontal angles with the direction method are identical to those of the NOS for 3rd order, Class I, while those given in GIHS specifications for traverse and triangulation are more relaxed than those of NOS. Another point about the British specifications is that they do not specify the length of traverse legs or triangulation baselines.

From this survey of the specifications of the various hydrographic agencies, several conclusions can be drawn.

The British specifications are generally the most strict, sometimes reaching extremes. For example, they require a 2.5 mm interval between intermediary soundings. In general, every agency employs standards which are equal to or better than those recommended by IHO. Of particular interest is the U.S. NOS use of root mean square error (drms) for the establishment of specifications concerning position accuracy.

From the examined hydrographic methods and techniques, of particular interest and value is the Canadian bar check method. The U.S. NOS method for sound velocity correctors, which combines the bar check and the oceanographic techniques, improves the quality of the final sound velocity correctors. In the area of automated hydrography, the Canadian example showed that microprocessors are probably the ideal solution for small launches.

In summary, the following suggestions are made to the Hellenic Navy Hydrographic Service which may increase and improve present productivity of the service:

(1) Develop detailed measurement procedures for horizontal control like those established by the U.S. NOS. Adopt the concept of confidence region used by the CHS to design and analyze surveys.

(2) Relax some of its strict hydrographic specifications in order to increase the present productivity. The maximum allowable sounding line spacing of 8 mm is one example. Both the U.S. NOS and the CHS have more relaxed requirements allowing 10 mm spacing.

(3) Develop some detailed specifications to meet the required standards for hydrographic positioning like those adopted by the U.S. NOS which are based on the use of root mean square error (drms).

(4) Adopt the Canadian bar check method for the determination of sound velocity correctors for launches. Supplement these with oceanographic observations similar to the U.S. NOS.

(5) Consider the use of microprocessors in future procurement and installation of automated systems, particularly in launches.

(6) Merge its existing hydrographic orders in accordance with the above recommendations to develop a contemporary and efficient hydrographic manual. It should cover the same material and have a general layout similar to that of the U.S. NOS Hydrographic Manual, which seems to be the most practical, complete and contemporary guide and reference source for both field and office work.

This survey and comparison of the standards and methods in hydrographic surveying of different countries, showed that the specifications and methods of each agency supplement those of each of the others. The surveyor can benefit greatly by being aware of the other methods so that he can modify and improve the methods he is traditionally accustomed to working with.

APPENDIX A

SUMMARY OF U.S. NATIONAL OCEAN SURVEY'S CLASSIFICATION STANDARDS OF ACCURACY AND GENERAL SPECIFICATIONS FOR HORIZONTAL CONTROL

Table XVII, taken from U.S. NOS specifications [Ref. 119], shows the classification, standards of accuracy and general specifications for horizontal control that are in use in the U.S. by the National Ocean Survey as well as by other federal agencies with surveying activities. From this table the columns of Third Order Class I and Class II are of particular interest for the hydrographic surveyor because these are the orders of accuracy he is usually required to accomplish.

The following explanations are necessary in order to understand the table:

(1) In the strength of figure section, R_1 and R_2 are values of R for the two best computational routes; the best computational routes are those having the least values.

(2) In the horizontal directions section, the instrument characteristic describes the recommended smallest reading of the horizontal circle of the theodolite used.

A position is one measure of the horizontal direction from the initial station to each of the other stations with the telescope both direct and reverse. This observational technique involves the selection of one signal as the initial. The circle and micrometer are set at a particular

TABLE XVII

CLASSIFICATION, STANDARDS OF ACCURACY, AND GENERAL SPECIFICATIONS FOR HORIZONTAL CONTROL

TRIANGULATION

Classification	*First-Order*	*Second-Order*		*Third-Order*	
		Class I	*Class II*	*Class I*	*Class II*
Recommended spacing of principal stations	Network stations seldom less than 15 km. Metropolitan surveys 3 km to 8 km and others as required.	Principal stations seldom less than 10 km. Other surveys 1 km to 3 km or as required.	Principal stations seldom less than 5 km or as required.	As required	As required
Strength of figure					
R₁ between bases					
Desirable limit	20	60	80	100	125
Maximum limit	25	80	120	130	175
Single figure					
Desirable limit					
R_1	5	10	15	25	25
R_2	10	30	70	80	120
Maximum limit					
R_1	10	25	25	40	50
R_2	15	60	100	120	170
Base measurement					
Standard error [1]	1 part in 1,000,000	1 part in 900,000	1 part in 800,000	1 part in 500,000	1 part in 250,000
Horizontal directions [2]					
Instrument	0".2	0."2	0".2 } or { 1".0	1".0	1".0
Number of positions	16	16	8 } or { 12	4	2
Rejection limit from mean	4"	4"	5" } or { 5"	5"	5"
Triangle closure					
Average not to exceed	1".0	1".2	2".0	3".0	5".0
Maximum seldom to exceed	3".0	3".0	5".0	5".0	10".0
Side checks					
In side equation test, average correction to direction not to exceed	0".3	0".4	0".6	0".8	2"
Astro azimuths [3]					
Spacing-figures	6-8	6-10	8-10	10-12	12-15
No. of obs./night	16	16	16	8	4
No. of nights	2	2	1	1	1
Standard error	0".45	0".45	0".6	0".8	3".0
Vertical angle observations [4]					
Number of and spread between observations	3 D/R—10"	3 D/R—10"	2 D/R—10"	2 D/R—10"	2 D/R—20"

TABLE XVII (continued)

TRAVERSE

Classification	*First-Order*	*Second-Order*		*Third-Order*	
		Class I	*Class II*	*Class I*	*Class II*
Recommended spacing of principal stations	Network stations 10-15 km Other surveys seldom less than 3 km.	Principal stations seldom less than 4 km except in metropolitan area surveys where the limitation is 0.3 km.	Principal stations seldom less than 2 km except in metropolitan area surveys where the limitation is 0.2 km.	Seldom less than 0.1 km in tertiary surveys in metropolitan area surveys. As required for other surveys.	
Horizontal directions or angles[5]					
Instrument	0".2	0".2 or 1".0	0".2 or 1".0	1".0	1".0
Number of observations	16	8 or 12*	6 or 8*	4	2
Rejection limit from mean	4"	4" or 5"	4" or 5"	5"	5"
Length measurements					
Standard error[1]	1 part in 600,000	1 part in 300,000	1 part in 120,000	1 part in 60,000	1 part in 30,000
Reciprocal vertical angle observations[4]					
Number of and spread between observations	3 D/R—10"	3 D/R—10"	2 D/R—10"	2 D/R—10"	2 D/R—20"
Number of stations between known elevations	4-6	6-8	8-10	10-15	15-20
Astro azimuths					
Number of courses between azimuth checks[7]	5-6	10-12	15-20	20-25	30-40
No. of obs./night	16	16	12	8	4
No. of nights	2	2	1	1	1
Standard error	0".45	0".45	1".5	3".0	8".0
Azimuth closure at azimuth check point not to exceed[8]	1".0 per station or 2' $\sqrt{N}$	1".5 per station or 3" $\sqrt{N}$. Metropolitan area surveys seldom to exceed 2".0 per station or 3" $\sqrt{N}$	2".0 per station or 6" $\sqrt{N}$. Metropolitan area surveys seldom to exceed 4".0 per station or 8" $\sqrt{N}$	3".0 per station or 10" $\sqrt{N}$. Metropolitan area surveys seldom to exceed 6".0 per station or 15" $\sqrt{N}$	8" per station or 30" $\sqrt{N}$
Position closure[9] after azimuth adjustment	0.04m $\sqrt{K}$ or 1:100,000	0.08m $\sqrt{K}$ or 1:50,000	0.2m $\sqrt{K}$ or 1:20,000	0.4m $\sqrt{K}$ or 1:10,000	0.8m $\sqrt{K}$ or 1:5,000

* May be reduced to 8 and 4, respectively, in metropolitan areas.

TABLE XVII (continued)

Number of figures between known elevations	4-6	6-8	8-10	10-15	15-20
Closure in length [8] (also position when applicable) after angle and side conditions have been satisfied, should not exceed	1 part in 100,000	1 part in 50,000	1 part in 20,000	1 part in 10,000	1 part in 5,000
		TRILATERATION			
Recommended spacing of principal stations	Network stations seldom less than 10 km. Other surveys seldom less than 3 km.	Principal stations seldom less than 10 km. Other surveys seldom less than 1 km.	Principal stations seldom less than 5 km For some surveys a spacing of 0.5 km between stations may be satisfactory.	Principal stations seldom less than 0.5 km.	Principal stations seldom less than 0.25 km.
Geometric configuration [9]					
Minimum angle contained within, not less than	25°	25°	20°	20°	15°
Length measurement					
Standard error [1]	1 part in 1,000,000	1 part in 750,000	1 part in 450,000	1 part in 250,000	1 part in 150,000
Vertical angle observations [4]					
Number of and spread between observations	3 D/R—10"	3 D/R—10"	2 D/R—10"	2 D/R—10"	2 D/R—20"
Number of figures between known elevations	4-6	6-8	8-10	10-15	15-20
Astro azimuths [5]					
Spacing-figures	6-8	6-10	8-10	10-12	12-15
No. of obs./night	16	16	16	8	4
No. of nights	2	2	1	1	1
Standard error	0".45	0".45	0".6	0".8	3".0
Closure in position [8] after geometric conditions have been satisfied should not exceed	1 part in 100,000	1 part in 50,000	1 part in 20,000	1 part in 10,000	1 part in 5,000

TABLE XVII (continued)

NOTE (1)

The standard error is to be estimated by

$$\sigma_m = \sqrt{\frac{\Sigma v^2}{n(n-1)}}$$

where σ_m is the standard error of the mean, v is a residual (that is, the difference between a measured length and the mean of all measured lengths of a line), and n is the number of measurements.

The term "standard error" used here is computed under the assumption that all errors are strictly random in nature. The true or actual error is a quantity that cannot be obtained exactly. It is the difference between the true value and the measured value. By correcting each measurement for every known source of systematic error, however, one may approach the true error. It is mandatory for any practitioner using these tables to reduce to a minimum the effect of all systematic and constant errors so that real accuracy may be obtained. (See page 267 of Coast and Geodetic Survey Special Publication No. 247, "Manual of Geodetic Triangulation," Revised edition, 1959, for definition of "actual error.")

NOTE (2)

The figure for "Instrument" describes the theodolite recommended in terms of the smallest reading of the horizontal circle. A position is one measure, with the telescope both direct and reversed, of the horizontal direction from the initial station to each of the other stations. See FGCC "Detailed Specifications" for number of observations and rejection limits when using transits.

NOTE (3)

The standard error for astronomic azimuths is computed with all observations considered equal in weight (with 75 percent of the total number of observations required on a single night) after application of a 5-second rejection limit from the mean for First- and Second-Order observations.

NOTE (4)

See FGCC "Detailed Specifications" on "Elevation of Horizontal Control Points" for further details. These elevations are intended to suffice for computations, adjustments, and broad mapping and control projects, not necessarily for vertical network elevations.

NOTE (5)

Unless the survey is in the form of a loop closing on itself, the position closures would depend largely on the constraints or established control in the adjustment. The extent of constraints and the actual relationship of the surveys can be obtained through either a review of the computations, or a minimally constrained adjustment of all work involved. The proportional accuracy or closure (i.e. 1/100,000) can be obtained by computing the difference between the computed value and the fixed value, and dividing this quantity by the length of the loop connecting the two points.

NOTE (6)

See FGCC "Detailed Specifications" on "Trilateration" for further details.

NOTE (7)

The number of azimuth courses for First-Order traverses are between Laplace azimuths. For other survey accuracies, the number of courses may be between Laplace azimuths and/or adjusted azimuths.

NOTE (8)

The expressions for closing errors in traverses are given in two forms. The expression containing the square root is designed for longer lines where higher proportional accuracy is required.

The formula that gives the smallest permissible closure should be used.

N is the number of stations for carrying azimuth.
K is the distance in kilometers.

value (recommended circle settings are given in Table XVIII). Each signal is then observed in a clockwise order and the results recorded. At the last signal, the telescope is reversed and the procedure is repeated in a counterclockwise order. The observed seconds for direct and reverse are meaned and the initial direction is subtracted from each observation referencing the measurements to an initial of $0^\circ 00' 00.00''$. The above procedure completes one position. To continue the observations, the above process is repeated for the next circle setting (taken from Table XVIII). Finally, the resulting measurements for each position are meaned and each angle is checked for the rejection limit from the mean.

The term "rejection limit from the mean" means that if angles at any position of the circle differ by more than this limit from the mean of the set, they must be reobserved before leaving the station. Triangle closure is the sum of the three observed angles of a triangle minus 180° minus the spherical excess[19].

A side equation is a series of length computations starting from a line, passing through successive triangles and finally returning to the starting line. A simple example is illustrated via Figure 31.

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[19] Spherical excess is the amount by which the sum of the three angles of a spherical triangle exceeds 180°.

TABLE XVIII

PLATE SETTING FOR THE HORIZONTAL OBSERVATIONS USING THE WILD T-2 AND THE KERN DKM-2 THEODOLITES

4 Positions

	°	'	"
1.	0	00	10
2.	45	02	40
3.	90	05	10
4.	135	07	40

8 Positions

	°	'	"
1.	0	00	10
2.	22	01	25
3.	45	02	40
4.	67	03	55
5.	90	05	10
6.	112	06	25
7.	135	07	40
8.	157	08	55

12 Positions

	°	'	"
1.	0	00	10
2.	15	01	50
3.	30	03	30
4.	45	05	10
5.	60	06	50
6.	75	08	30
7.	90	00	10
8.	105	01	50
9.	120	03	30
10.	135	05	10
11.	150	06	50
12.	165	08	30

16 Positions

	°	'	"		°	'	"
1.	0	00	10	9.	90	00	10
2.	11	01	25	10.	101	01	25
3.	22	02	40	11.	112	02	40
4.	33	03	55	12.	123	03	55
5.	45	05	10	13.	135	05	10
6.	56	06	25	14.	146	06	25
7.	67	07	40	15.	157	07	40
8.	78	08	55	16.	168	08	55

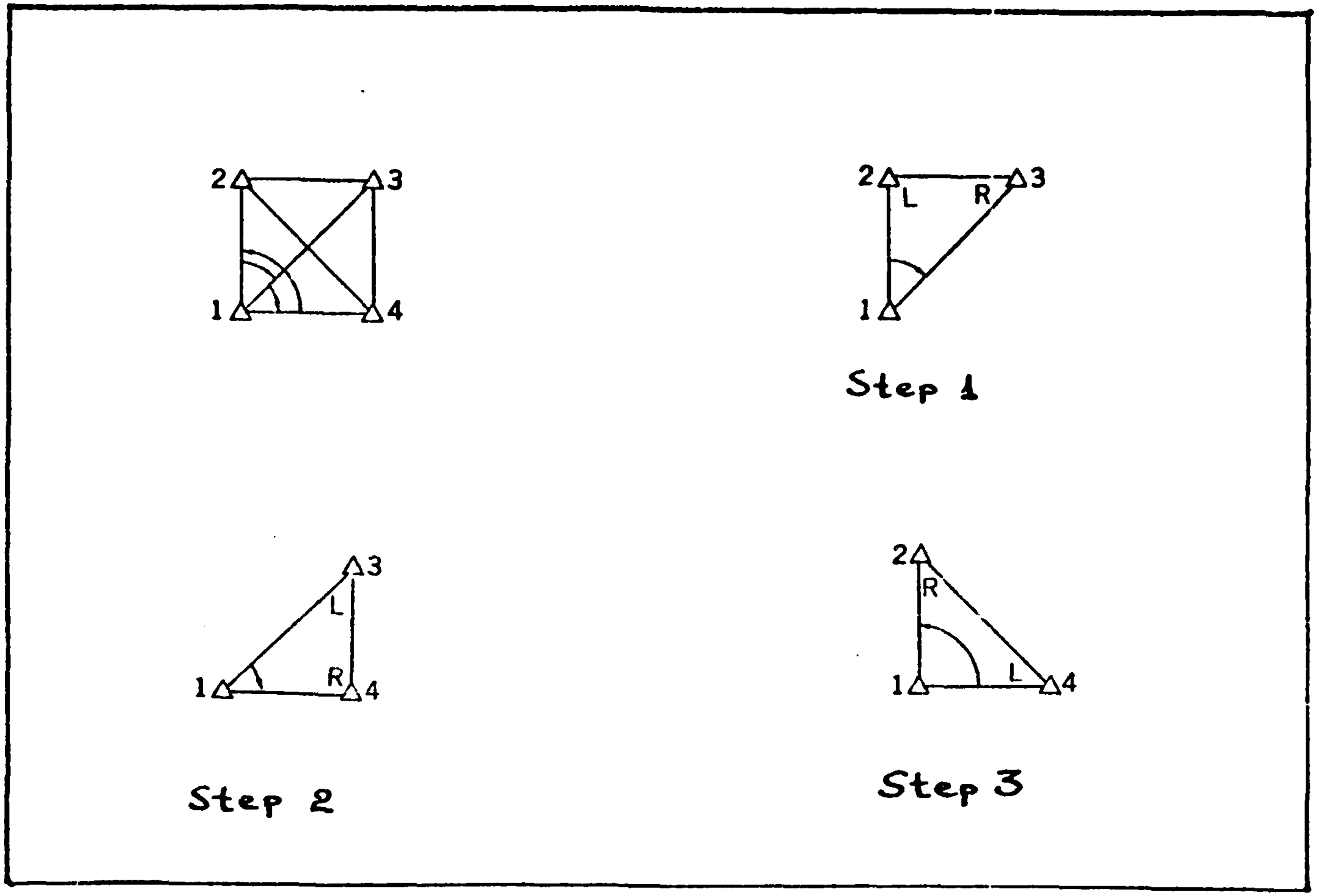

Figure 31. Side Check Steps in Triangulation

<u>Step 1</u> - Starting with the line 1-2, the line 1-3 is computed (by the law of sines).

<u>Step 2</u> - Now from the determined line 1-3, the line 1-4 is computed.

<u>Step 3</u> - In the same manner the line 1-2 is computed from the previously determined line 1-4.

The discrepancy in the sides is the difference between the value of side 1-2 determined in Step 3 and the starting value, For side equation tests, the actual length of the starting line can be ignored, and assumed to be 1 or any other arbitrary value since this value will appear in both compared values because the law of sines has been used through the computational route.

In order to obtain the average correction to an angle in seconds of arc and compare it with the values given in Table XVII we use the formula:

$$T = \frac{\frac{1}{2} S \rho''}{\sum|\cot L| + \sum|\cot R|}$$

where: T is the correction in seconds of arc.

is the number of seconds per radian = 206264.8.

$\sum|\cot L|$ is the sum of the absolute values of cotangents of the left angles.

$\sum|\cot R|$ is the same as above sum but for the right angles.

where: Π stands for the product of the sines of left or right angles.

Left or right angles (see Figure 31) are determined by the principle that left angles are thos opposite known sides while right angles are those opposite the unknown sides. The equations are tabulated as shown in Table XIX.

TABLE XIX

SIDE CHECK EQUATION EXAMPLE (SEE FIGURE 31)

	Observed Left Angles	sines	$\|cot\|$	Observed Right Angles	sines	$\|cot\|$
Step 1	70° 21' 09.8"	0.9717664	0.24280	40° 20' 18.0"	0.64729989	1.17756
Step 2	34° 34' 11.6"	0.56860811	1.44671	89° 16' 29.3"	0.99991990	0.01266
Step 3	32° 57' 02.1"	0.54391549	1.54278	27° 40' 03.6"	0.46434232	1.90733

Product of sines

$\pi \sin L = 0.30054290$ $\qquad$ $\pi \sin R = 0.30054466$

Sum of $|\cot|$

$\Sigma|\cot L| = 3.23228$ $\qquad$ $\Sigma|\cot R| = 3.09755$

$$S = \frac{\pi \sin L}{\pi \sin R} - 1 = \frac{0.30054290}{0.30054466} - 1 = -5.868 \times 10^{-6}$$

$$T = \frac{1/2\ Se''}{\Sigma|\cot L| + \Sigma|\cot R|} = \frac{1/2 \times (-5.868 \times 10^{-6}) \times 206264.8}{(3.23228 + 3.09755)} = -0.1''$$

Average correction per angle is -0.1"

APPENDIX B

EXAMPLES OF STANDARD DEVIATIONS FOR VARIOUS INSTRUMENTS AND METHODS

From: "Specifications and Recommendations for Control Surveys and Survey Markers," Energy, Mines and Resources, Canada, 1978.

TABLE XX

LENGTH BY TAPE AND SUBTENSE BAR

METHOD	STANDARD DEVIATION (metres)	REMARKS
Invar Tape	$\sqrt{0.003^2 + (0.3\,L\,10^{-6})^2}$	Techniques described in Geodetic Survey Pub. 73. L = line length in metres.
Steel Tape	$\sqrt{N\,(0.002^2 + (40\,P\,10^{-6})^2)}$	N = number of tape lengths. P = length of each tape in metres. Very careful slope, sag and temp. corrs. applied.
Steel Tape	$\sqrt{N\,(0.006^2 + (80\,P\,10^{-6})^2)}$	N and P as above. Clinometer used for vertical angles up to 5°; alignment by picket; air temp. used for corrections; tension handle used for spans over 30 m.
Steel Tape	$\sqrt{N\,(0.01^2 + (200\,P\,10^{-6})^2)}$	N and P as above; no tension handles; nominal temp. correction.
Subtense Bar	$\sqrt{2\,(0.001)^2 + (2.5\,L^2\,10^{-6})^2}$	Standard deviations of approx. 1" for angle measurements and of 1 mm for plumbing at each end of line. L = line length in metres.

TABLE XXI

LENGTH BY VARIOUS TYPES OF EDM INSTRUMENTS

Note: The proportional part of the accuracy of all EDM measurements is limited by the accuracy of the modulation frequency, by the accuracy of the meteorological measurements and by the reliability of the samples measured as representative of the conditions along the measured line. The non-proportional part is dependent on the accuracies of plumbing and of the *zero* corrections of the instrument and reflector and the resolution of the instrument. The modulation frequency and zero correction of the instrument must be checked whenever the instrument is repaired, or after rough treatment as well as on a regular schedule. Accuracy standards for recording meteorological measurements, for frequency of calibration tests and for elevation differences are given in Table XXII and are referred to in this table.

Type	Standard Deviation (metres)	Standard*	Remarks
Mekometer	$\sqrt{0.0002^2 + (2.0\,L\,10^{-6})^2}$	M1	Forced centering.
Tellurometer MA100 (infrared)	$\sqrt{0.005^2 + (3\,L\,10^{-6})^2}$	M2	Mean of 8 measurements anytime.
	$\sqrt{0.010^2 + (5\,L\,10^{-6})^2}$	M5	Mean of 2 measurements anytime.
Ranger 3 and 4 (Laser)	$\sqrt{0.010^2 + (2\,L\,10^{-6})^2}$	M2	Mean of 8 measurements anytime.
	$\sqrt{0.015^2 + (4\,L\,10^{-6})^2}$	M5	Mean of 2 measurements anytime.
Geodimeter (visible light)	$\sqrt{0.010^2 + (1.2\,L\,10^{-6})^2}$	M2	Mean of 6 measurements.
	$\sqrt{0.030^2 + (3.6\,L\,10^{-6})^2}$	M5	Mean of 2 measurements.
Tellurometer MRA2	$\sqrt{0.05^2 + (3L10^{-6})^2}$	M3	37 cavities, three double measurments.**
	$\sqrt{0.11^2 + (8\,L\,10^{-6})^2}$	M4	10 cavities, single measurement.
Tellurometer MRA3	$\sqrt{0.03^2 + (3\,L\,10^{-6})^2}$	M3	13 cavities, three double measurements.**
	$\sqrt{0.08^2 + (8\,L\,10^{-6})^2}$	M4	10 cavities, single measurement.
Tellurometer MRA4	$\sqrt{0.015^2 + (3\,L\,10^{-6})^2}$	M3	10 cavities, three double measurements.**
	$\sqrt{0.06^2 + (8\,L\,10^{-6})^2}$	M4	10 cavities, single measurement.

L = line length in metres.
*Standards as listed in Table XXII.

**Successive measurements alternating master end. Successive doubles separated by at least four hours.

TABLE XXII

STANDARDS FOR METEOROLOGICAL MEASUREMENTS, FOR CALIBRATION TESTS AND FOR ELEVATION DIFFERENCES AND MEAN ELEVATION DETERMINATIONS

Standard*	Meteorological Measurements				Calib. Schedule (additional to calib. after repair or rough usage)		Elevation Determinations	
		Standard Deviations					Standard Deviations for a line L metres long	
	Temp. at one or both ends	Pressure (mm)	Dry Bulb Temp. (°C)	Wet Bulb Temp. (°C)	Zero	Freq.	Elev. Diff h** (m)	Mean Elev. H (m)
M1	both	3	0.1	0.1	2 mo.	2 mo.	$\frac{L}{h}(0.0002 + 10^{-6}L)$	2
M2	both	3	0.2	0.2	4 mo.	1 mo.	$\frac{L}{h}(0.003 + 10^{-6}L)$	2
M3	both	3	0.2	0.2	4 mo.	2 wks.	$\frac{L}{h}(0.01 + 2.10^{-6}L)$	2
M4	both	3	0.5	0.5	2 yr.	6 mo.	$\frac{L}{h}(0.02 + 10^{-6}L)$	10
M5	one only	3	1	—	6 mo.	1 yr.	$\frac{L}{h}(0.005 + 3.10^{-6}L)$	10

*These standards are those referred to in Table XXI

**For example: to achieve the accuracies listed in Table XXI for measurements requiring standard M2, for a line 1000 m long between terminals 25 m different in elevation, the elevation difference should be measured by a method giving a standard deviation not greater than:

$$\frac{1000}{25}(0.003 + 0.001) = 0.16 \text{ m.}$$

TABLE XXIII

DIRECTIONS

Instrument Least Count	STANDARD DEVIATIONS		Remarks
	Seconds	Metres perp. to L	
0.2"	$\sqrt{\left(\frac{3 \times 10^5 P}{L}\right)^2 + (0.6)^2}$	$\sqrt{2P^2 + (3.0L \times 10^{-6})^2}$	L = Line length in metres. P is std. dev. of plumbing of instrument and target. Normal values: 0.0014m. Four groups of 6 sets during 2 days. Starting time of each group differing by at least 2 hours.
1.0"	$\sqrt{\left(\frac{3 \times 10^5 P}{L}\right)^2 + 1.5^2}$	$\sqrt{2P^2 + (7.5L \times 10^{-6})^2}$	Mean of 6 sets.
1.0"	$\sqrt{\left(\frac{3 \times 10^5 P}{L}\right)^2 + 2.0^2}$	$\sqrt{2P^2 + (10L \times 10^{-6})^2}$	Mean of 4 sets.
1.0"	$\sqrt{\left(\frac{3 \times 10^5 P}{L}\right)^2 + 4^2}$	$\sqrt{2P^2 + (20L \times 10^{-6})^2}$	Mean of 2 sets.
10.0"	$\sqrt{\left(\frac{3 \times 10^5 P}{L}\right)^2 + 8^2}$	$\sqrt{2P^2 + (40L \times 10^{-6})^2}$	Mean of 2 sets.
20.0"	$\sqrt{\left(\frac{3 \times 10^5 P}{L}\right)^2 + 15^2}$	$\sqrt{2P^2 + (75L \times 10^{-6})^2}$	Mean of 2 sets.
30.0"	$\sqrt{\left(\frac{3 \times 10^5 P}{L}\right)^2 + 21^2}$	$\sqrt{2P^2 + (100L \times 10^{-6})^2}$	Mean of 2 sets.

Note: The angle between two directions with standard deviations σ_1 and σ_2 as derived above will have a standard deviation $\sqrt{\sigma_1^2 + \sigma_2^2}$. If the lengths of the lines are about the same in the most accurate case above, ($\sigma_1 = \sigma_2$), the standard deviation of the angle between them will be about:

$$\sqrt{\left(\frac{4.2 \times 10^5}{L} P\right)^2 + (0.85)^2} \text{ seconds.}$$

TABLE XXIV

AZIMUTHS

INSTRUMENTS	STD. DEV.*	REMARKS
First-Order Astronomic	1"	Azimuth and longitude observed.
Second-Order Survey Theodolite with stride level (Polaris observations)	3"	No observed longitude. Non-mountainous terrain where *small deflections in prime vertical* can be expected and estimated or assumed zero.
Second-Order Survey Theodolite without stride level (Polaris observations)	6"	
Second-Order Survey Theodolite with stride level (Polaris observations)	about 10"	No observed longitude. *Mountainous terrain where large deflections can* be expected and which can only be roughly estimated.
Second-Order Survey Theodolite without stride level (Polaris observations)	about 15"	
Gyro Theodolite	3" - 10"	No observed longitude. Using special techniques. Ultimate accuracy depends on local deflection.
Gyro Theodolite	20"	No observed longitude. Using normal techniques.
Second-Order Survey Theodolite (solar observations)	10" - 20"	With Roelofs Solar Prism, depending on method.
	30" - 45"	Without Roelofs Solar Prism, depending on method.

*Standard deviation of azimuth transferred to a line between two stations.

TABLE XXV

POSITION DIFFERENCES

Method	Standard Deviation	Remarks
Satellite Doppler	0.5 m	50 passes over at least 48 hours; broadcast or precise ephemeris; simultaneous observations at two or more stations; simultaneous (multi-station) solution for positions and orbital biases.
Satellite Doppler	1.5 m	50 passes; precise ephemeris; non-simultaneous observations at the two stations.
Satellite Doppler	7.5 m	50 passes; broadcast ephemeris; non-simultaneous observations at the two stations.
Inertial Survey System (Litton Autosurveyor)	0.5 m	Double run in straight line by helicopter between control spaced at 80 km.

Notes: *Position Differences by Satellite Doppler*

Differences of geographic position between two or more stations may be obtained by Doppler observations of satellites. The values in this table refer to observations of the United States Navy Navigation Satellite System, also known as the TRANSIT system. Application of the precise ephemeris to the solution will give a position whose standard deviation relative to datum is 1 m, whereas for the broadcast ephemeris this standard deviation is 5 m. When only the broadcast ephemeris is used, two known stations must be observed simultaneously with each group of unknown stations; this is to provide adequate accuracy and orientation relative to both datum and existing stations. In either case (broadcast or precise ephemeris) a receiver with internal timing for Doppler counts will give best results. The standard deviation of a 30 second Doppler count should not exceed 20 cm. Meteorological observations (temperature, pressure and relative humidity) should be taken every six hours (more frequent during weather changes such as *front*) for use in the tropospheric refraction correction model. Two days (48 hours) of observing are needed to provide a desirable range of meteorological conditions. Also, 48 hours of observing is needed to provide the complete range of satellite to ground geometry. Ionospheric refraction effects are largely removed through the use of two frequencies. Receivers should be warmed up at least three days before measurement, and antennae should be mounted at survey tripod height over surfaces that are not radio-reflective.

Satellite passes whose approach elevation angle is less than 15° at closest approach should not be used. Also Doppler counts observed below 7.5° should not be used because of the inadequacy of the tropospheric refraction correction model near the horizon.

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